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FRANK·A·RANDALL

Randall

AMERICAN CONCRETE INSTITUTE

PROCEEDINGS

OF THE

TWENTY-FIRST ANNUAL CONVENTION

Held at Chicago, Ill.
February 24, 25, 26 and 27, 1925

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AMERICAN CONCRETE INSTITUTE

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G-5.—Wason Medal.

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J-1.—Joint Committee on Concrete and Reinforced Concrete—Institute Representation.

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J-2.—Institute Representation on Joint Culvert Pipe Committee.

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E-1.—Reinforced-Concrete Building Design and Specifications.

F. R. McMillan, *Chairman.*
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A. E. Lindau, George E. Horr, Manton E. Hibbs, A. W. Stephens, A. R. Lord, H. N. Howe, H. P. Mueller, W. A. Slater, R. B. Young, W. S. Thomson, H. M. Westergaard, L. C. Urquhart, Carl Preis, F. E. Richart, S. C. Hollister, H. J. Brunnier, R. L. Bertin, H. D. Loring, F. C. Wight, I. Kvitrud.

E-3.—Research.

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E-4.—Fire Resistance of Concrete.

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R. R. Zipprodt, M. J. O'Brien, Leonard C. Wason.

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R. W. Crum, *Chairman.*

R. L. Bertin, J. G. Ahlers, N. M. Loney, Duff A. Abrams, F. H. Jackson, H. S. Mattimore, H. J. Love, L. R. Ferguson, H. E. Breed, H. F. Clemmer.

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F. R. McMillan, *Secretary.*

P. J. Freeman, R. B. Young, G. E. Warren, W. K. Hatt, E. Viens.

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A. C. Irwin, *Secretary.*

J. E. Freeman, W. F. Kearns, Charles R. Miller, W. M. Rynerson, A. C. Tozzer, H. R. Wood, T. A. Pearson.

C-3.—TREATMENT OF CONCRETE SURFACES.

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F. A. HITCHCOCK, *Secretary.*

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C-5.—MEASUREMENT OF AND ESTIMATING CONCRETE.

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S-1.—REINFORCED-CONCRETE CHIMNEYS.

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S-2.—REINFORCED-CONCRETE HIGHWAY BRIDGES AND CULVERTS.

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LANGDON PEARSE, *Chairman.*

M. W. LOVING, *Secretary.*

W. L. LEA, F. A. MARSTON, C. J. REQUARDT, C. E. DE LEUW, ASA E. PHILLIPS, E. S. RANKIN.

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S-6.—CONCRETE ROADS AND PAVEMENTS.

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C. R. EGE, *Secretary.*

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P-1.—STANDARD BUILDING UNITS.

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C. C. WOODY, KENT MERRY, L. H. DEMUTH.

P-5.—FIRE RESISTANCE OF CONCRETE BUILDING UNITS.

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HARVEY WHIPPLE, *Secretary.*

W. R. HARRIS, S. H. INGBERG, H. J. LOVE, F. W. LUMIS, A. R. SMALL, G. M. FRIEL, STANTON WALKER, JOHN H. MCCLATCHY, AUSTIN CRABBS, J. J. BLISS.

P-6.—CONCRETE PRODUCTS PLANT OPERATION.

J. W. LOWELL, *Chairman.*

C. L. BOURNE, *Secretary.*

W. D. M. ALLAN, A. G. SWANSON, JACOB BOSCH, G. J. LENGST, J. J. BLISS, C. E. LINDSLEY, BERT CAREY, GEORGE SAFFERT, DAVIS EWING, W. H. CAREY, AUSTIN CRABBS, H. F. GONNERMAN, E. H. HILKER, LAWRENCE COX, E. W. DIENHART, C. L. DOUTHETT.

P-7.—PIPE, DRAIN, TILE, AND CONDUIT.

C. F. BUENTE, *Chairman.*

M. W. LOVING, *Secretary.*

ERNEST F. BENT, M. L. ENGER, J. E. LONGLEY, R. S. LANDER, J. A. DUNN, J. S. LAMBIE, C. H. BULLEN, SAMUEL FORTIER, PAUL KIRCHER, B. S. PEASE, WILLIS WHITED, E. S. RANKIN.

P-8.—CONCRETE ROOF TILE.

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BY-LAWS.

AMERICAN CONCRETE INSTITUTE.

ARTICLE I.

MEMBERS.

SECTION 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

SEC. 2. A firm or company shall be treated as a single member.

SEC. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

SEC. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at the time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

SEC. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

ARTICLE II.

OFFICERS.

SECTION 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

SEC. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

Sec. 3. There shall be a Committee of five members on Nomination of Officers, elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall elect by letter ballot of its members, candidates for the various offices to become vacant at the next Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

Sec. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-President and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

Sec. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members

on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

SEC. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President on the first regular session of the annual convention.

SEC. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

SEC. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

SEC. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

SEC. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

SEC. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

SEC. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III.

MEETINGS.

SECTION 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of Convention.

SEC. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

SEC. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

SEC. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

ARTICLE IV.

DUES.

SECTION 1. The fiscal year shall commence July 1st.

SEC. 2. The annual dues shall be ten dollars ($10.00) payable annually in advance from first of the month following notification of the applicant of his election by the Board of Direction.

SEC. 3. Each member shall be entitled to receive one copy of one volume of the Proceedings for each membership year and additional volumes at a price fixed by the Board of Direction.

SEC. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon payment of all indebtedness against them upon the books of the Institute.

ARTICLE V.

STANDARDS.

SECTION 1. Proposed new or revised Standard Specifications, Standard Practice and Standard Definitions, when approved by a majority voting in the committee in which they originate, shall be submitted in the form adopted in the Standard Form of Standards to the secretary of the Institute 60 days prior to the opening of the annual convention at which they are to be presented. The secretary of the Institute shall cause these proposed new standards or revised standards to be printed as Proposed Tentative Standards and mailed to the full membership of the Institute thirty days prior to the opening of the convention. As there amended and approved, they shall be published in the Annual Proceedings, next issued as Tentative Standards. At a subsequent annual convention they may again be offered unamended, by their originating committees as proposed standards, and as there approved by a majority of those voting, they shall be submitted to letter ballot of the Institute membership, to be canvassed within ninety days thereafter. Such proposed standards shall be considered adopted unless at least 10 per cent of the total membership shall vote in the negative.

ARTICLE VI.

AMENDMENT.

SECTION 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF PROCEEDINGS OF THE TWENTY-FIRST ANNUAL CONVENTION.

Drake Hotel, Chicago, Illinois.

First Session, Tuesday, February 24, 1925, 2 P. M.

The convention was called to order by A. E. Lindau, president of the American Concrete Institute.

The following papers were read and discussed:

"Theory Must Aid Practice in Concrete Making," by C. P. Richardson.

"The Design of Concrete Mixtures Under Field Conditions," by T. P. Watson.

"Field Control of Concrete on the Delaware River Bridge," by A. W. Munsell.

"Notes on Laitance," by R. M. Miller.

Joint Second Sessions, Tuesday, February 24, 1925, 8 P. M.

Two co-ordinate sessions were held on Tuesday evening (a) for those interested in reinforced-concrete building construction, with Vice-President E. D. Boyer in the chair, and (b) for concrete products manufacturers, with A. J. R. Curtis in the chair. The meeting consisted of the discussion of certain questions previously printed.

Third Session, Wednesday, February 25, 1925, 9.30 A. M.

President A. E. Lindau in the chair.

The report of Committee E-6, "Destructive Agents and Protective Treatments," was presented by F. R. McMillan, secretary. The report was discussed.

The report of Committee E-5, "Aggregates," was presented by Frank H. Jackson, secretary. The report was a progress report, and was accepted.

The report of the Institute's representation on the Joint Committee on Concrete and Reinforced Concrete (Committee J-1) was presented by S. C. Hollister, chairman.

The motion was adopted that the report of the Joint Committee on Concrete and Reinforced Concrete be received by the Institute and printed in the *Proceedings.*

The report of Committee E-1, "Building Laws," was presented by F. R. McMillan, chairman. The report consisted in the presentation of preliminary studies for revised standard building regulations.

FOURTH SESSION, WEDNESDAY, FEBRUARY 25, 1925, 2 P. M.

Mr. J. W. Lowell in the chair.

This meeting was devoted to a discussion of progress in details of operation of plants for the manufacture of concrete building units—a discussion to cover the broad work of Committee P-6, on Concrete Products Plant Operation. It consisted of a discussion led by Mr. Lowell and C. L. Bourne, chairman and secretary of Committee P-6, respectively.

At the meeting there was consideration of Committee P-6 report on Recommended Practice for the Manufacture of Concrete Building Block, Building Tile, and Brick.

FIFTH SESSION, WEDNESDAY, FEBRUARY 25, 1925, 8 P. M.

Past-president Richard L. Humphrey in the chair.

The following papers were read and discussed:

"Concrete from the Viewpoint of Mr. Cement," by Thaddeus Merriman.

"Notes on the Construction of a Concrete Stadium," by W. H. Hatt.

"Crazing on Cement Products," by P. H. Bates.

(A presentation by the chairman of Committee P-1 on Crazing and his understanding of the problems of that newly-formed committee.)

"Shall Anything Be Added to Portland Cement," by Maximilian Toch.

"Proportioning Concrete Materials with Special Reference to Highway Construction," by G. W. Hutchinson.

"Coefficient of Expansion Tests on Gunite," by M. O. Fuller (by title only).

SIXTH SESSION, THURSDAY, FEBRUARY 26, 1925, 9.30 A. M.

Past-president Richard L. Humphrey in the chair.

Report of Committee S-6 on Roads and Streets was read by C. R. Ege, secretary. The report consisted in the presentation of Tentative Standard Specifications for One-Course Concrete Pavement for Streets; Tentative Standard Specifications for Two-Course Concrete Pavement for Streets, Tentative Standard Specifications for Two-Course Concrete Pavements for Highways.

All were adopted by the meeting to be sent to letter ballot for adoption as standards.

The following paper was read and discussed:

"Applying the Inundation Method to Control Moisture Content," by A. A. Levison.

The report of Committee P-7 on Pipe and Drain Tile was presented by C. F. Buente, chairman. The report presented revisions on Tentative Specifications for Concrete Drain Tile, Plain Concrete Sewer Pipe and Reinforced-Concrete Sewer Pipe.

The report made the following recommendations: That the Tentative Standard Specifications for Concrete Drain Tile P-7B-24T be submitted to letter ballot of the Institute and become a standard (adopted by the meeting). (2) That the Proposed Standard Specifications for Plain and Concrete Drain Tile, 1910-1911, be withdrawn (adopted by the meeting). (3) That the Recommended Practice for Plain Concrete Drain Tile of 1912 and Standard Specifications for Concrete Drain Tile of 1917 be submitted to the Institute by letter ballot for withdrawal (adopted by the meeting). (4) That Tentative Standard Specifications for Reinforced-Concrete Sewer Pipe P-7C-24T be adopted as Tentative Standards by the Institute.

The following paper was read and discussed:

"Finishing Concrete Floors," by E. E. Davis.

The report of Committee C-2 on Floors and Sidewalks was read by A. C. Irwin, secretary. The report consisted in the recommendation that Tentative Standard Specifications for Portland Cement Concrete Sidewalks C 2B-24T be advanced to Standard (adopted by the meeting). (2) That there be a revision of Standard Specifications for Concrete Floors C-2A-24 (motion was adopted).

The following papers were read and discussed:

"Central Mixing Plants," by W. E. Hart.

"Securing Texture for Portland Cement Stucco," by Samuel Warren.

SEVENTH SESSION, THURSDAY, FEBRUARY 26, 1925, 2 P. M.

President A. E. Lindau in the chair.

The report of the Board of Direction was read by the Secretary.

Past-president Richard L. Humphrey in the chair.

The tellers announced the following elections:

President, A. E. Lindau, of Chicago.
Vice-President (2-year term), Edward D. Boyer, New York.
Secretary and Treasurer, Harvey Whipple, Detroit.
Directors, 3rd District, S. C. Hollister.
4th District, John J. Earley.
5th District, Duff A. Abrams.

President Lindau in the Chair.

A discussion on the suggestions for furthering the work of the American Concrete Institute followed.

The report of Committee C-3, "Treatment of Concrete Surfaces," was presented by J. C. Pearson, chairman.

A new Tentative Standard Recommended Practice for Treatment of Exterior Surfaces of Industrial Reinforced-Concrete Buildings was presented and was ordered by the meeting to be continued into Tentative Standards.

The committee further presented progress report, which was received.

The report of Committee S-5 on Concrete Dwelling Houses was presented by J. A. Ferguson, chairman. The report consisted in proposed Recommended Practice for the Design and Construction of Concrete Dwelling Houses. It was accepted as a progress report.

The report of Committee P-1 on Concrete Standard Building Units was presented by C. L. Bourne, secretary.

Tentative Standard Specifications for Concrete Building Block and Concrete Building Tile, and for Concrete Brick were submitted to be advanced to Standards.

The report of Committee P-5 on Fire Resistance of Concrete Building Units was presented by Leslie H. Allen, chairman. The report was accepted.

The report of Committee P-6, "Concrete Products Plant Operation," was presented by J. W. Lowell, chairman. It presented Proposed Revised Recommended Practice for the Manufacture of Concrete Building Block, Building Tile and Brick. Accepted as tentative.

Annual Banquet, Drake Hotel, Chicago, Ill., Feb. 26, 1925, 7 P. M.

Frank C. Wight, toastmaster.

Addresses were made by A. E. Lindau, president; James Weber Linn, professor of english at the University of Chicago; and F. R. Moulton, professor of astronomy, University of Chicago.

Frank C. Wight, chairman, Wason Medal Committee, presented the Wason Medal to Richard L. Humphrey.

EIGHTH SESSION, FRIDAY, FEBRUARY 27, 1925, 9.30 A. M.

President A. E. Lindau in the chair.

The report of Committee C-5, "Measurement of and Estimating Concrete," was presented by Frank R. Walker, chairman. It contained Tentative Specifications for the Measurement of and Estimating of Concrete, which were adopted as such.

The report of Committee S-1, "Reinforced-Concrete Chimneys," was presented by Eric Plagwit, chairman. The report consisted in the preliminary report on Recommended Practice for the Design of Reinforced-Concrete Chimneys and Proposed Tentative Specifications for the Construction of Reinforced-Concrete Chimneys. After discussion, the report was referred back to the committee for further study.

The following paper was read and discussed:

> "Report of Tests Made to Determine the Temperatures in Reinforced-Concrete Chimney Shells," by E. A. Dockstader.

The report of Committee G-4 on Nomenclature was presented by W. A. Slater. The report was accepted as a progress report.

The report of Committee E-4 on Fire Resistance of Concrete was presented by N. D. Mitchell, chairman. It was a progress report accepted by the meeting.

The report of Committee E-8 on Expansion Joints was presented by title.

The following paper was read and discussed:

> "Design of Reinforced-Concrete Circular Bins for the Storage of Cement," by H. A. Ward.

THE WASON MEDAL.

Awarded Each Year to the Author of the Most Meritorious Paper Presented to the Previous Annual Convention.

Awarded 1925 to

Richard L. Humphrey, for two papers, "Twenty Years of Concrete" and "The Promise of Future Development," Presented to the 1924 Convention.

Previous Awards.

1916 Convention Paper—A. B. McDaniel, "Influence of Temperature on the Strength of Concrete."

1917 Convention Paper—Charles R. Gow, "History and Present Status of the Concrete Pile Industry."

1918 Convention Paper—Duff A. Abrams, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919 Convention Paper—W. A. Slater, "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

1920 Convention Paper—W. A. Hull, "Fire Tests of Concrete Columns."

1921 Convention Paper—H. M. Westergaard, "Moments and Stresses in Slabs."

1922 Convention Paper—George E. Beggs, "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gauges."

1923 Convention Paper—J. J. Earley, "Building the Fountain of Time."

THE TASKS BEFORE US.

OPENING ADDRESS BY A. E. LINDAU.*

It is my privilege, gentlemen, to call this meeting to order, to extend to you a word of welcome, and formally to introduce to you our twenty-first annual convention. Our convention culminates the year's work; it measures, roughly, the annual output of our organization; and functions as a clearing house for the exchange of information and ideas of mutual interest. We are organized to serve the concrete industry in its many activities by providing an open forum where our common problems may be fully and freely discussed. By highly specialized committee work, particular problems are solved, suitable specifications prepared, and rules of practice recommended.

The work of the Institute being entirely voluntary, it is gratifying and also surprising to find that year after year a large number of our members will give unsparingly of their time and energy to share with us their knowledge and experience. When the burden of carrying on this work, in addition to the daily routine, seems irksome, when we are face to face with the question as to whether all this effort is worth while, we are apt to forget that the rate of progress in our field, as in every other, is in some proportion to the speed with which we communicate or interchange our information. The ever surprising rate at which we are adding to our knowledge and advancing the boundaries of science can be accounted for to a large extent by co-operation in effort and improvement in our mode of communication. As we meet here annually, our increment of experience is added to the general fund, thereby making it available to our industry as well as to the community as a whole.

At our last convention we turned our attention particularly to our achievements in the past, and more especially to the progress of the past twenty years. We have reason to feel proud of that record. A marvelous amount of work has been done. Our volumes of proceedings constitute a storehouse of information that is invaluable to all who are interested in concrete. Going through these volumes it would seem as though we had mastered our main problems, and that from now on we can concern ourselves principally with standardization of practice and refinement of method. We are tempted to pause on our journey even though we realize that we are not at our destination. But, we may only pause to catch our breath: we are not permitted to stand still, but must resume the journey. As we begin our second stage of twenty years, what are the obstacles in our way? What are some of the tasks immediately before us?

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*President, American Concrete Institute.

An immense fund of information has been accumulated regarding the properties of concrete, its resistance to the elements, its behavior under stress when subjected to external forces or load. By a vast amount of laboratory and field research, we have discovered many fundamental laws governing the making of a satisfactory product. But to what extent is all this information being utilized? It will be admitted, I think, that at the present time we are storing up data faster than we are making use of it. A visit to concrete jobs under construction will reveal the fact that the making of the concrete, the most important feature in concrete construction, does not generally indicate the advance that has been made in our knowledge of the matter.

One of our difficulties, no doubt, lies in the inertia of trade practice. It has become the custom to produce concrete along certain accepted lines. The equipment and tools available, the handling and character of materials have been adjusted to trade custom and are not readily changed or seriously modified without protest.

This particular problem, then, seems to be one of education, of continuing to spread broadcast the things that have already been learned. It is not a simple nor an easy matter. We are confronted with the task of modifying firmly established practices and habits of thought, of compelling attention to the possibilities of making better concrete, all along the line, from the designer of the structure to the man in charge of the mixing crew. Indeed, lack of knowledge concerning concrete is not confined to the humble workman who actually handles the materials, but is frequently found among those who sit in the "seats of the mighty." This is not a reflection on those in charge of our structure, but rather a statement of the extent of the problem. The building industry has become exceedingly complex and highly specialized. By specialization we have become expert in a particular field, but this specialization has its drawbacks and disadvantages. The many questions that confront the architect or engineer in producing plans and specifications as well as supervising the construction of the structure, absorb his attention to such an extent that it is difficult for him to become expert in the knowledge of all the materials he uses. To him concrete is just one of the materials he requires. But concrete differs from other materials in that it is manufactured in the field, it cannot be accepted or rejected at the place of manufacture. Each job in itself becomes a manufacturing plant, and the quality of the resulting product subject in a large degree to the varying conditions under which it is produced.

It seems clear, then, that one of the tasks immediately before us is to reduce our knowledge of making good concrete to its simplest terms and carry that message to the man in the office as well as the man on the job. We hear a great deal these days of revival in craftsmanship, it seems to be a reaction to the continued demand for quantity production. But we are in the grip of quantity production, our economic situation seems to require it, and therefore it is not likely that this condition will

change. However, educating the man on the job holds out a promise of at least a partial solution of the problem, by enlisting the interest and attention of the workman in *quality* production.

In line with our campaign of education, a very promising step has been taken in the matter of field control of concrete. Although considerable work has been done in this direction, the subject is new and the technique is undeveloped. At this moment it would be rash to predict how far these new methods will carry us toward the goal of better concrete. They do, however, carry the results of our research directly to the job and stimulate interest in obtaining a quality product.

One of our tasks is to encourage this practice where practicable, to aid in developing its possibilities, and formulate, if we can, a satisfactory technique.

We have attributed to concrete the quality of permanence to such an extent that the words are almost synonymous. Examples of concrete construction that have lasted since the time of the Roman Empire have been cited to illustrate the enduring quality of this material. But we are learning that there are conditions under which concrete may not be permanent. Many reasons have been advanced to account for this situation. Investigations have been made; especially of structures subjected to particularly trying conditions. In some cases the main cause for the unsatisfactory situation could be clearly and definitely determined, while in others the causes are obscure. The investigation of a structure that shows symptoms of deterioration is naturally surrounded with some difficulties. Data that would be helpful is frequently not available—the publication of the facts impracticable for various reasons. And yet it is obviously of the greatest importance that a thorough study be made of the subject in all its phases in the hope that the destructive agencies may be isolated and satisfactory remedies applied.

As we look about us we find on all sides work to be done, tasks already undertaken requiring completion, new areas that invite exploration. It may not be profitable to attempt to arrange these tasks in the order of their importance. Their importance changes with changing conditions. We have, for example, not considered it of first importance to give much thought to the properties of cement itself. It is generally agreed that our present product, when properly used, meets our requirements admirably. But there are indications that in our restless search for improvement, cement is about to be placed under observation and further study. The result of research here may carry in its train a whole series of new problems and newer tasks to be accomplished. This situation, however, seems to lie some distance ahead. The matter of most urgent importance is to devise ways and means of obtaining quality concrete in our structures. When the task has been done there is reason to believe that one of the main causes of deterioration has been removed and that concrete will maintain the reputation it has acquired, namely, that of being permanent.

Theory Must Aid Practice in Concrete Making.

By C. P. Richardson.*

The statement: "Theory must aid practice in concrete making," chosen as the subject of this paper, will be approved by the average engineer; he will, however, have a very marked mental reservation as to just how far he will "go along" with this principle.

He knows that a lot of excellent concrete was made before we ever heard of "scientific concrete," scientific field control, Abram's water-cement ratio, Captain Edwards surface area theory, Professor Talbot's voids-cement theory, or other theories which have been advanced thus far. He also believes that a lot more good concrete will be made without any definite knowledge of these theories. He generally overlooks the fact, however, that he is constantly making use, although unknowingly, of principles which have been established in part by practicable field experience and in part by laboratory research. In other words, better concrete at the same cost, or better concrete at less cost, is being made through the use of correctly interpreted principles and practices obtained through scientific research.

No experienced concrete foreman will allow mixtures to be used that are too harsh to be easily workable. He knows that fine sand requires more cement and more water to obtain the consistency which he desires for the particular work in hand. He has learned that, within the limits of a workable consistency which will eliminate "honey comb," the coarser his stone and the more of it he uses, the sounder and harder the concrete. The influence of water on the strength and density was known in an imperfect way by practical men before the relations of strength and the ratio of water to cement was developed by Professor Abrams. However, practice in general was farther away from scientific knowledge in this respect than in any other factor affecting the quality of concrete. Many other "rule of thumb" or practical principles developed by experience could be cited, which, unknown to the practical man, are now recognized through scientific research as fundamental to successful concrete work.

Practical knowledge and experience will always be necessary in successfully applying any fundamental theory of concrete making. On the other hand, a knowledge of this fundamental theory will enable the practical man to cease groping for practices to overcome defects whose cause he does not know. It sets limits for him beyond which he may not go, provides gauges not heretofore available by which he may measure the

*Engineer of Track Elevation, Chicago, Rock Island & Pacific Ry., Chicago.

effect of changes in practice to meet varying conditions. In fact, theory has now placed the practical man's feet on the ground, released him from many uncertainties and given him the opportunity to apply his practical knowledge with confidence, and to develop his practice along definite lines.

A personal experience along these lines may serve to illustrate the leading thought I have in mind. Not over ten years ago I was resident engineer on a project demanding the rapid deposit of large quantities of mass concrete. The work consisted, for the most part, of large sections of relatively high retaining walls which were poured each day. To make for speed the time of mixing was reduced to a minimum; to make a flowing or workable mixture water was added. The resultant concrete and surface finish of the walls was far from satisfactory. With the usual rigid adherence to the standard specified proportions of cement and fine and coarse aggregates experiments were confined to increasing the time of mixing and to adding or lessening the amount of water, but the results were always far from satisfactory. To make the narrative brief, I can say unquestionably that, had information as to the present-day methods been available at that time, there would have been much less experimenting and much better concrete obtained at less cost, the trouble in this particular case being unusually wet and coarse sand and a strict adherence to the standard proportions of aggregates without regard to an analysis of the combined aggregates used on this work.

It is not my purpose in this short paper to attempt a complete discussion of any of the theories of proportioning now available, but, rather to outline some of the practical problems involved in their application, briefly mention the progress made by railroad engineers and suggest some ways that the concrete wizards can further enlighten the average engineer.

The first practicable problem encountered is to understand the theories advanced by various investigators. Without such an understanding, the man who is to apply the theory must follow specific rules, the relative importance of which he cannot judge. Anyone who has had experience with concrete work knows the ever-changing variety of conditions confronting him. None of the factors affecting the quality of the concrete is constant. The aggregates vary in grading and in maximum size. The moisture in the aggregate changes from day to day and with successive shipments. The ease or difficulty of getting the concrete from the mixer into the forms is almost never the same, even on the same job. In order that the practical man may cope with these conditions he must be informed as to the fundamental principles of concrete mixtures. Simplification of the scientific expositions coming from laboratories and concrete experts is necessary before we can expect the full advantage of research about concrete. Papers like those on this program will do much to make these seemingly involved theories understandable by the common, practical man. I do not mean by this to criticise laboratory publications as such. They are evidently written largely from the investigator's point of view and to a

certain extent to forestall attack from other investigators. The engineer or practical man is not so much interested in attacking a theory as he is in understanding its practical value. He simply isn't interested in it, if he can't understand it or put it to good use. The first thing to do, therefore, is to reduce a theory that is to be applied in practice by a great number of non-technical men to its very lowest terms. I believe this is best done by making clear the fundamental principle or law of the theory.

Of the various important theories which have been advanced with a view of bettering the making of concrete, I want to draw particular attention to the water-cement ratio which has been thoroughly developed by Prof. Duff A. Abrams. This simple theory, I believe, marks the way for better concrete by serving as the ground work with which all other theories can be utilized further to amplify its value.

As I understand it, the fundamental principle of the water-cement ratio is that *within the limit of workable mixes, the strength of concrete bears a definite relation to the ratio of the total volume of water in the mix to the volume of cement used.* All other factors such as grading of aggregates (fineness modulus), fineness of sand, total aggregates to cement, etc., depend for their influence on strength, to their influence on the ratio of water and cement necessary to make a plastic mixture. That is, nothing has a vitally important bearing on the strength of the concrete except as it affects the ratio of the water to the cement that must be used for the work in hand. This suggests that in all discussions of Abrams' theory we should start with this fundamental truth and trace the effect of each other factor on this ratio. It has already been advanced and with good reason that a practical man can mix his concrete to any desired plasticity within the limits of his experience as to what will make satisfactory concrete, by varying the proportions of aggregates as his judgment dictates *without altering the strength of the resulting concrete, so long as he does not change the ratio of the total water to the cement.* This gives us something to start with—something that a foreman can understand and work with. Of course there is the unknown factor of the amount of moisture already in the aggregate, but a drying pan, a measure and a pair of scales are all that is necessary to find out how much this is. Obviously, the greater amount of aggregate used with no change in the amount of water the dryer the consistency and the greater the yield. Tests have been made especially designed to check up whether a constant strength will be obtained by keeping the water constant, and changing the aggregate at will, so long as the limits of proper plasticity are not overstepped. These tests show practically no change in strength over a wide range of proportions.

We must, therefore, conclude that arbitrary proportions such as 1:2:4 have no meaning insofar as the strength of the concrete is concerned if the total quantity of water used is not kept constant. Strengths varying as much as 100 per cent may be had with the same apparent proportions of cement and aggregate by changing the water ratio. Obviously, therefore,

a third constituent should be added to the proportions for concrete, namely, water. The proportions would then be stated cement, water and aggregate, and of these three, the most important is water.

In addition to the direct effect of water on the strength of concrete, it has an indirect effect, the importance of which has not been fully appreciated by concrete users throughout all the years that concrete has been extensively used. I refer to the bulking effect of moisture in sand. It is known that *all* sands suitable for concrete bulk or expand as moisture is added, up to a certain percentage of contained moisture. This bulking or expansion amounts to as much as 40 per cent maximum for many sands and is almost always so great as to make field measurements far from what is expected. Thus, if the sand is dry, ten wheelbarrows of it will give much more actual sand than will ten wheelbarrows of sand that has bulked 25 per cent or 30 per cent. In other words, 25 to 30 per cent *less sand* will be used in the mix if it is measured when wet rather than when dry. Practical men have often been bothered to explain why they had sand left over, or why the cement used did not check up with yield of concrete obtained. An answer is found in the bulking of the fine aggregate.

It seems worth while, here, to point out that the customary published tables of quantities of cement and aggregates are based on a measure of aggregates under standard conditions and can only give rough approximations of the actual quantities required when measured as received in the field. If the total aggregate vary 20 per cent in the amount used, even when the field measuring is so well done as to give uniform apparent volumes, how can such tables have any reliable meaning? Not only do standard proportions, such as 1:2:4, have no meaning, but, also, the usual tables of quantities have no reliability.

Concrete experts should produce tables that are based on the scientific knowledge we now have about concrete; but tables should also be made that can easily be understood and used with simple corrections relative to the variable factors. As data accumulate as to the effect of these variable factors, they can be analyzed, tabulated and made available for the use of contractors and estimators.

As a general rule, railroad concrete work is carried on at widely distant points and in quantities varying from a few to thousands of cubic yards. Some of it is done by company forces and some by contractors. The first consideration of the railroad engineer, in planning any construction, is safety. Rigid and detailed specifications constitute one of the methods of the railroad engineer in securing the kind of work he wants. Field control methods should be included in these specifications, but to be of any value they must be of such a nature that they can be easily interpreted by contractors and construction forces. Such a specification may be in existence, but I have never seen it. Probably the only way such a specification can be obtained to take care of railroad conditions is through the railroad engineers themselves. Specifications and instruc-

tions prepared by a committee of this Institute, composed of railroad engineers, would undoubtedly be best adapted to the conditions of railroad construction. The work described in papers presented today, together with the experience gained on a number of other railroads, form a sufficient background for an intelligent approach to the problem. Scientific field control methods on the Sidney bridge of the Big Four Railroad, the Rigolets Pass bridge of the L. & N., the Newark Bay Bridge of the Central Railroad of New Jersey, were described in proceedings of the Institute, 1923. Full test data and discussion of the latter are contained in the January *Proceedings* of the American Society of Civil Engineers. In addition to these, several other railroads have gone more or less into field control methods.

The Northern Pacific Railway Co., New York Central, the Canadian National (Western Region) and other lines have taken definite steps to standardize methods of making concrete. The railroads are not alone in taking steps to establish more perfect methods in their concrete work, but the references cited indicate considerable progress being made by them.

Detailed reports on the use of modern scientific principles in connection with important projects are now available. We are greatly indebted to the engineers who have done this work, and without reservation on their part have presented to the engineering profession detailed information as to the methods used, the various problems involved, and the results that were obtained.

I find, however, that the average engineer evidences a considerable hesitancy in adopting these methods on his particular project. He is inclined to question their practicability, due to a belief that their use demands a greatly increased cost of inspection and supervision which could perhaps be better invested in additional cement. He can also cite instances of the use of additional cement as a result of these newer methods. He fails to appreciate, however, that the true reason for this increase is due to the fact that, by the use of these principles, the engineers' attention is generally directed to the strength of the concrete, and he often finds that he is not producing the quality of concrete that he had planned for that class of work. With the product failing to meet the designed strength, there are times when the more economical method is to increase the cement content rather than lessen the water and make a change in the aggregates which are available on the particular project. On the other hand, if concrete is being produced with a strength as demanded by the adopted design stresses, the same modern methods show the way to reduce the cost of this concrete by a simple adjustment of the water content and the relative mixture of the aggregates with the resultant saving in cement and in the cost of the concrete as a whole.

There is no question in the mind of the speaker but that the use of modern scientific principles does result in better concrete at the same cost or an equal quality of concrete at a less cost, and by directing attention to the quality or the strength of the concrete being produced under the

usual field conditions, will go a long way toward the bettering of the concrete work of the future.

The average engineer, however, will continue to believe that these principles are too complicated or involved, until such time as the methods utilized in the laboratory work can better point the way to more simple methods which are practicable under normal field conditions.

With the data, which have been obtained through experience from many sources, analyzed, sorted and arranged, those of us who have to work out a way to overcome our own particular problems, could be saved a lot of time. No one who is not over-conservative, will contend that we will continue to go along as we have been, in the practice of making concrete. Neither will anyone who has spent hours, days and weeks, trying to find out what these scientific theories are all about, contend that he has been properly treated by those who have done the scientific research.

Theory must aid practice by being practical, by recognizing the limitations and preconceived ideas of the practical man. Theory must talk the language of the practical man, must take itself to his work in the field, must understand what can and what cannot be done with Bill and John who do the work, must reduce itself to processes that are so simple and safeguarded that a "hunkey" of average intelligence cannot go wrong. Theory must produce for us tables, constants, factors, etc., for use in simple arithmetic computations. It must aid in the development of equipment and machinery that will carry out its laws expeditiously, economically and with certainty.

When this is done, practice will follow the correct theory, better concrete will be produced, great economies obtained and an added impetus given to the already increasing use of concrete in all branches of construction work.

DISCUSSION.

Mr. Humphrey. RICHARD L. HUMPHREY.—I was interested in Mr. Richardson's remarks that for railroad work it will be necessary to have a committee of railroad engineers to prepare specifications for their work. What is there so difficult about railroad work that specifications that are amply satisfactory for other work could not be used in railroad work?

Mr. Richardson. MR. RICHARDSON.—The idea I intended to convey is that railroad work consists of a large number of relatively small pieces of work distributed over a large territory where inspection and supervision are minor considerations at the present time. To make use of these more modern principles on those smaller jobs is a problem which I myself am hardly able to pass upon. I believe that it should be looked into and possibly some very simple rules could be established that could be practically put into use on those widely distributed jobs.

Mr. Humphrey. MR. HUMPHREY.—I have an idea that there are other small jobs on which the hundred and thirty or forty million barrels of portland cement are used besides large public structures; in fact, the bulk of our cement is used on small jobs. Now the need of better concrete on large jobs that we can control is the first step in getting after the use of better concrete on the small jobs, and I think we will agree that there are very few of the large jobs so far that have put in those methods, so there is plenty of opportunity for work. Now, when it comes to the small jobs, it would seem to me that instead of having a committee of railroad engineers, you ought to have a committee made up of representatives of that type of work and see how those rules that are applicable to the large job should be modified or what other additional rules are necessary to take care of controlling the concrete on the small jobs.

Mr. Lindau. MR. LINDAU.—It would seem to the chair that the railroad work offers an opportunity for taking care of the small jobs that perhaps the average run of work does not offer, insofar that it might be possible for a railroad to train a crew in the making of good concrete and keep that crew busy over a large territory if they do their work themselves. Of course when a railroad contracts for work which they would do in the more important cases rather than small jobs, that is more difficult to handle.

Concrete Mixtures Under Field Conditions.

By T. P. Watson.*

The general interest shown in methods to improve the quality of concrete is proof that we are going to profit by the rather unsavory past and at last are ready to admit there is something to be learned and accomplished by a more rational attitude toward the essential requirements which are necessary, if we are to obtain the best results. Why have we had concrete failures? We have had concrete failures in the vast majority of cases because of lax or incompetent supervision and inspection during construction and not on account of faulty design or materials.

We must have competent supervision and inspection to have good concrete work. You can not supervise a concrete job from an office by reading daily reports and progress photographs. The place to supervise a concrete job is on the ground and supervision should be so arranged that the only duties of concrete inspectors are the inspection of the mixing and placing of concrete. The concrete inspector is a vital factor in the life of a concrete structure. He should be a man with practical experience and "concrete sense" and one who will personally supervise the placing of every batch of concrete. Don't load him up with clerical work filling in forms and reports. I have never known a good inspector who did not find it burdensome to do any kind of clerical work. Arrange to have someone else make up reports and permit the concrete inspector to attend to his real duties and his time will be well taken up.

We have just completed a reinforced-concrete structure in Pittsburgh, known as the Beck's Run Bridge. This project involved the placing of 5,000 cu. yd. of mass concrete and 10,000 cu. yd. of reinforced concrete. We feel that the results we have obtained will be a lasting monument to a "real concrete inspector," who was present when every batch of concrete was placed in this structure.

All the concrete for this project was designed for a definite 28-day strength using the theory of "Design of Concrete Mixtures" by Prof. Duff A. Abrams. We designed mixtures for varying strengths from a 2,000 lb. minimum strength to 4,000 lb. maximum strength, using all the available commercial sizes of local river sand and gravel. We used the various sized aggregates to prove out the theory under actual working conditions. Regardless of the size of the aggregates used, we obtained uniform strengths using proportions in accordance with Prof. Abrams' theory. There is a

*Former Assistant Engineer, Engineering Department, Pennsylvania R. R. System.

marked economy in the use of cement by using the larger-sized coarse aggregates so long as they are graded down to the smaller sizes.

We experienced a wide variation in the workability of the concrete using different sands. Sand with a maximum size of ⅛ in. made much more plastic and workable concrete than sands with larger maximum sizes. I will outline as briefly as possible our method of designing concrete for a specified strength using as a basis Bulletins 1 and 9 of the Structural Materials Research Laboratory, Lewis Institute, Chicago. The proportions are of a unit of cement to units of dry rodded sand and coarse aggregates. Sand and gravel loose and moist, as they were placed in the concrete mixer on our work, contained various percentages of moisture and bulked accordingly. This was especially true of the sand so that in order to have uniform amounts of sand in our concrete mixtures it was essential to make determinations of the ratio of the volume of the loose materials to the volume of dry rodded materials.

Our first procedure was to obtain representative samples of the sand and gravel to be used in the concrete and to determine the moisture content and the bulking of each. For these determinations we used a cylindrical metal receptacle with a cubical capacity of ½ cu. ft. We filled this receptacle with the sample of loose sand and weighed it. We then dried this sand and weighed it. The difference in the weight of the moist sand and the dry sand was the moisture content. We then filled the receptacle with dry sand in three equal layers, rodding each layer twenty-five times with a ¾-in. diameter tamping rod and leveled off the top of the sand with the fingers. By dividing the weight of the dry rodded sand in the full receptacle by the weight of the dry sand of our sample, we had determined the ratio of bulking of the loose moist sand to a unit of sand dry and rodded. The same procedure was followed in determining the moisture in the gravel and its bulking. By a series of bulking tests of Pittsburgh gravel of various sizes we found that the gravel bulks 8 per cent or for our purpose we considered 108 per cent as constant, the volume of loose gravel to a unit of gravel dry and rodded.

We next made a sieve analysis of the sand and gravel separately, using a set of United States Standard square mesh sieves whose sizes are 100, 50, 30, 16, 8 and 4 per linear inch and ⅜ in., ¾ in., 1½ in. and 3 in. We used a 50-oz. sample of the dried sand and a 100-oz. sample of the dry gravel in making our sieve analysis. The percentage of the particles of the aggregate by weight retained on each sieve was obtained and the sum of these percentages divided by 100 gave us the fineness modulus.

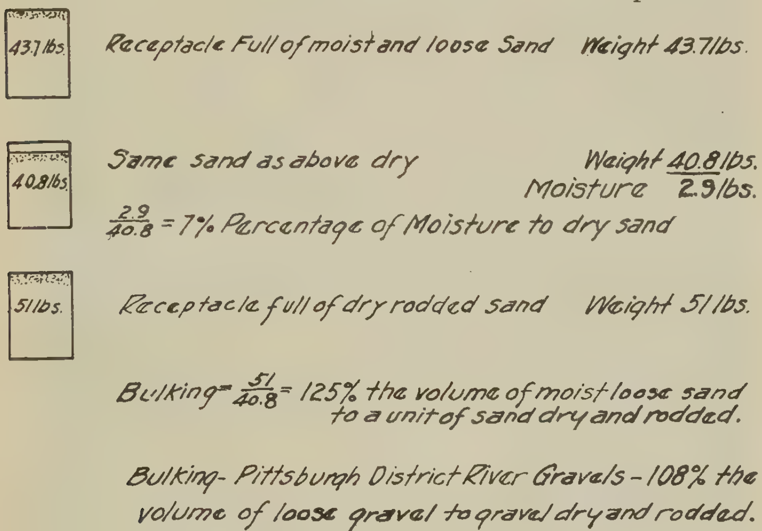
To illustrate, we will assume that the sieve analyses of the sand and gravel were as follows:

Size of Sieve	100	50	30	16	8	4	⅜	¾	1½	3	F.M.
Sand	99	88	30	15	8	1	0	0	0	0	2.41
Gravel	100	99.7	99.4	98.8	98.3	96.9	91.7	64.7	26.0	0	7.75

We now have determined the following:

Bulking of loose sand	125 per cent
Bulking of loose gravel	108 per cent
Fineness modulus of sand	2.41
Fineness modulus of gravel	7.75
Size of sand	0 to No. 8
Size of gravel	⅜ to 2 in.

The size classification of aggregates is determined from the sieve analysis. At least 15 per cent shall be retained on the sieve next smaller than that considered the maximum size and not more than 15 per cent of a



DETERMINING BULKING PER CENT.

coarse aggregate shall be finer than the sieve considered as the minimum size.

For instance, a graded sand with 15 per cent retained on the No. 8 sieve would fall in the 0-No. 4 size; if 14 per cent or less were retained, the sand would fall in the 0-No. 8 size. A coarse aggregate having 16 per cent coarser than the 2-in. sieve would be considered as 3-in. aggregate. The 1-in. and 2-in. sieves are not used in determining the fineness modulus of coarse aggregate but should be used for the size classification. We now have sufficient data to design concrete of any strength desired with the very important exception of the water required, which element I will take up later.

Our next step was to decide on the workability or plasticity of the concrete required for the particular part of the structure to be poured. In other words, workability or plasticity should mean the use of the driest

concrete possible consistent with common sense. At Beck's Run we used the following consistencies and obtained excellent results. For gravity sections where it was possible for men to be in the forms we used a slump of 3 in. For reinforced concrete where it was not possible for men to be in the forms but where it was possible for men to puddle the concrete with long-handled hoes and paddles we used a 4-in. slump. For reinforced-concrete where it was impossible for men to be in the form and impossible to reach the concrete with any practical puddling device we used a 6-in. slump. We then decided on the strength required.

To illustrate, we will assume that we are pouring a gravity section abutment with 2,000 lb. concrete required and that we will use a 3-in. slump.

We have found by the sieve test that the sand size was 0 to No. 8 and the gravel size ⅜ in. to 2 in. By referring to the tables in Bulletin 9

1 cu.ft. of sand dry and rodded — Weight 102 lbs.
1 cu.ft. of gravel dry and rodded — Weight 104 lbs.

Mix-Bulletin- 1.0:2.2:4.8

2.2 cu.ft. of sand dry and rodded @ 102 lbs. = 224.4 lbs.
4.8 cu.ft. of gravel dry and rodded @ 104 lbs. = 499.2 lbs.
7.0 cu.ft. by volume separated = 723.6 lbs.

$\frac{2.2}{7.0}$ = 31% Percentage of sand in mix.

1 cu.ft. of a mixture of aggregates dry and rodded containing 31% sand and 69% gravel = 120.7 lbs.

7.0 × 120.7 = 844.9

Yield = $\frac{723.6}{844.9}$ = 85.7%

True Mix = 7.0 × 85.7% = 6.0 cu.ft. of combined dry and rodded aggregates to 1 sack of cement (1 cu.ft.) or True Mix = 1:6.0

PROCESS OF TRUE MIX DETERMINATION.

for 2,000 lb. concrete, we find under fine aggregates size 0 to No. 8 and coarse aggregate size ⅜ in. to 2 in. for a 3 to 4-in. slump, a proportion of 1 cu. ft. of cement (one sack) to 2.2 cu. ft. of sand dry and rodded and 4.8 cu. ft. of coarse aggregate dry and rodded. This proportion with the proper amount of water could be used were it not for certain necessary corrections which I will explain later.

The Abrams theory of mix is based on what is termed the true mix, or the proportion between a cubic foot of cement (one sack) and units of dry sand and dry coarse aggregates combined and rodded. If we combine and thoroughly mix the 2.2 cu. ft. of sand and the 4.8 cu. ft. of gravel of our example and would place them in a receptacle, the resultant volume would be less than the volume of the two aggregates measured separately as the sand would fill in the voids in the gravel.

VIEW OF ONE OF THE COMPLETED PIERS.

The concrete was poured from a mixer which discharged about 15 ft. above the top of the pier into trunk chutes that reached down to the level of the concrete in the forms. The sections of the chute were removed as the level of the concrete raised. Note the lines of pier and the condition of the surface and corners. The concrete in these piers was poured with a 6-in. slump using coarse aggregate of a maximum size of three-quarters of an inch and designed for 2,000 lb. strength.

By the results of a series of determinations using the Pittsburgh aggregates we found that this decrease in volume was 86 per cent of the separated units of sand and gravel in varying proportions. Therefore the true mix of our example would be the proportion of 1 cu. ft. of cement (one sack) to 86 per cent of the sum of 2.2 and 4.8 or 6.02. The true mix then is 1 cu. ft. of cement to 6 cu. ft. of combined sand and gravel dry and rodded.

To facilitate the operations as outlined on a previous page, we made tests and determined the average weight of a cubic foot of sand dry and

VIEW SHOWS METHOD OF CONSTRUCTION.

Traffic discontinued on two tracks of the old structure from which the new piers were built complete. The viaduct supporting the abandoned tracks was then razed and three concrete girders and one-half of the concrete slab completed. Traffic was then transferred to two tracks on the finished half of the structure and the remaining old viaduct removed and the last half of the bridge completed.

rodded and a cubic foot of gravel dry and rodded. We then made determinations for the weights of a cubic foot of mixtures of sand and gravel dry and rodded. This we did by making mixtures of 10 per cent sand and 90 per cent gravel, 30 per cent sand and 70 per cent gravel, 40 per cent sand and 60 per cent gravel, etc., and ascertained the unit weights of each mixture dry and rodded. Having these unit weights the determination of the yield is a very simple proposition. Unit weights should be determined

of aggregates for the various sizes from each source of supply as the aggregates from different sections of the country have varying properties peculiar to themselves and their weights are likely to be quite different.

I referred to necessary corrections to be made to the proportions taken from the tables in Bulletin 9. Our reasons for these corrections were:

1. That the grading of the gravel received on the work was so variable that it was necessary to actually design a mixture for each carload of aggregate used in order to get uniform concrete. For instance, we had carloads of gravel of which 30 per cent would pass a No. 4 sieve.

2. The Pittsburgh district gravel contains a large proportion of flat particles and in order to secure the proper amount of mortar in the concrete it was necessary to increase slightly the sand and decrease the quantity of gravel.

Mr. Abrams in Bulletin 1, Table 3, has compiled a table of "Maximum Values of Fineness Modulus of Aggregates," together with corrections for various materials and states "that for pebbles consisting of flat particles reduce the values shown in Table 3 by twenty-five one-hundredth (0.25)."

We have obtained very satisfactory results by following this suggestion.

In order to take care of these corrections it was necessary, as stated before, to redesign the proportions taken from Bulletin 9. This we did by using the values of fineness modulus of aggregates in Bulletin 1, Table 3.

Earlier in the design of our mixture we found the true mix to be 1 to 6 and by our sieve test we found that the maximum size of our aggregate was 2 in. so that the size of our combined mixture of sand and gravel would be from 0 to 2 in.

In Bulletin 1, Table 3, we find values of fineness modulus of aggregates sized from 0 to 2 inches and a mix of 1 to 6 to be 6.05. Making a deduction of twenty-five one-hundredths for the flat particles as explained before we obtain the fineness modulus of our mix 5.80.

We are now ready to correct our proportions. This is done by using a formula and the aggregates of our example as follows:

$$P = 100 \, \frac{A - B}{A - C}$$

Where

P = percentage of fine aggregate in total mix.
A = Fineness modulus of coarse aggregate (7.75)
B = " " " final aggregate mixture (5.80)
C = " " " fine aggregate (2.41)

or

$$P = 100 \, \frac{7.75 - 5.80}{7.75 - 2.41} = 36,$$ the percentage of sand in the total mixture.

We found the total mixture was 2.2 and 4.8 to make a true mix of 1 to 6. Therefore 36 per cent of the sum of 2.2 and 4.8 gives us 2.52

cu. ft. of sand and the difference between 2.52 and 7 is 4.48, the cubage of the gravel.

We now have the proportion of 1 cu. ft. of cement to 2.52 cu. ft. of sand and 4.48 cu. ft. of gravel but it is still necessary to take another step to determine what volume of loose materials we are to place in the mixture

TAKING THE TEMPERATURE OF THE CONCRETE.

as the proportion 2.52 cu. ft. of sand and 4.48 cu. ft. of gravel is for volumes dry and rodded.

Our first determination showed us that the sand loose bulked 125 per cent and the gravel loose bulked 108 per cent, therefore by multiplying 2.52 x 125 per cent we obtain 3.15 cu. ft. of loose sand and by multiplying 4.48 x 108 per cent we obtain 4.84 cu. ft. of loose gravel. We now have the proportions of the volumes of cement and aggregates to place in the

concrete mixer to be 1 sack of cement, 3.15 cu. ft. of loose sand and 4.84 cu. ft. of loose gravel. Knowing these proportions the size of the batch is determined by the capacity of the mixer.

Now these proportions would make 2,000 lb. concrete *providing* the last and one of the most important ingredients is added in the proper quantity.

We gauged the water content entirely by the slump test. We did this by assuming that an estimated amount of water would give us the desired slump when we started in the morning and we would make at least two slump tests to check up our estimated water content. If these slumps

MAKING A SLUMP TEST.

A slump test is a method of control to prevent the use of too much water in concrete. A slump test is made by using a metal receptacle shaped like a frustum of a cone, 4 in. in diameter at the top, 8 in. at the bottom and 12 in. high and a ⅝ in. blunt pointed rod 21 in. long. Fill the receptacle with the concrete to be tested in layers approximately 4 in. deep and rod each layer exactly 30 times. Then immediately lift the receptacle vertically and carefully, and measure the settlement of the truncated cone of concrete from its original height by placing the receptacle alongside the concrete and measuring with a rule the settlement or slump of the concrete.

were about 3 in., we then placed the same quantity of water in each batch. Throughout the day if nothing unusual developed in the working of the concrete, we would make four or five additional slump tests to check the water. If our inspector noted any change in the uniformity of the concrete, that is, if it appeared too wet or too dry he immediately made slump tests to cut down the water if the slump was over 3 in. or to add additional water if the slump was under 3 in.

Our gaging of water by the slump test using Pittsburgh river aggregates up to a maximum size of 2 in. has been checked against the theoretical quantity of water in accordance with Bulletin 1 and we have found that the water content as determined by the slump test was almost

exactly the same as the theoretical water content. We made 6 x 12 in. test cylinders in accordance with standard practice and had them crushed at a commercial laboratory at the expiration of twenty-eight days. One of these test cylinders was taken from each 40 cu. yd. of concrete placed in the superstructure and the results of these tests prove conclusively the soundness of this theory of designing concrete mixtures.

There are a few essential points I would like to bring out in connection with the placing of concrete at the Beck's Run Bridge which are relevant.

We did not permit chuting from a central plant.

We insisted on two minutes' actual mixing time after all the ingredients were in the mixer.

We covered all horizontal surfaces with 3 in. of sand as soon as the concrete had set sufficiently to permit it and we kept the sand moist by sprinkling it twice daily to aid the proper curing of the concrete.

All the cement used on the work was furnished by the railroad company and was tested for 28-day strength before shipment from the cement mill.

We made colorimetric tests of all sand used in accordance with the standard test for organic impurities in sand and concrete of the American Society for Testing Materials.

We had a mixer in good physical condition.

We had an accurate batching device for measuring separately the sand and gravel placed in each batch.

We made an accurate measurement of the water added to each batch.

The contract for this work was let before we knew anything about this method of designing concrete mixtures and I do not think that the carrying on of this method in any way increased the contractor's expense.

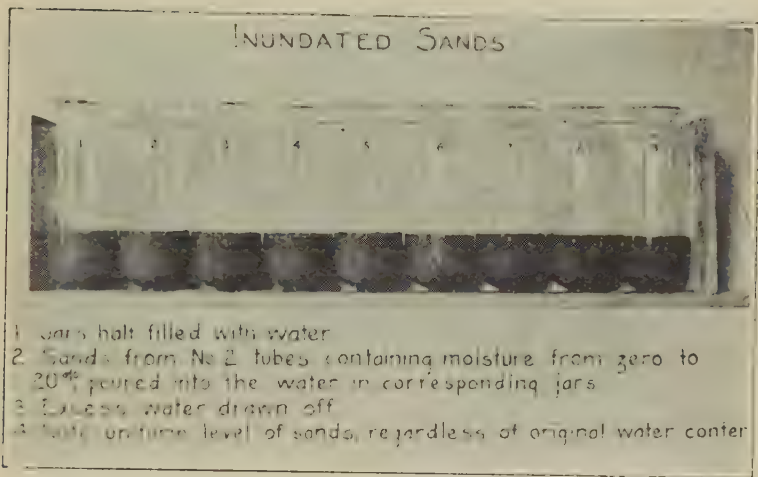
There was no additional expense to the railroad company, as the same engineering force would have been employed on this work even though we had mixed our concrete by the old methods.

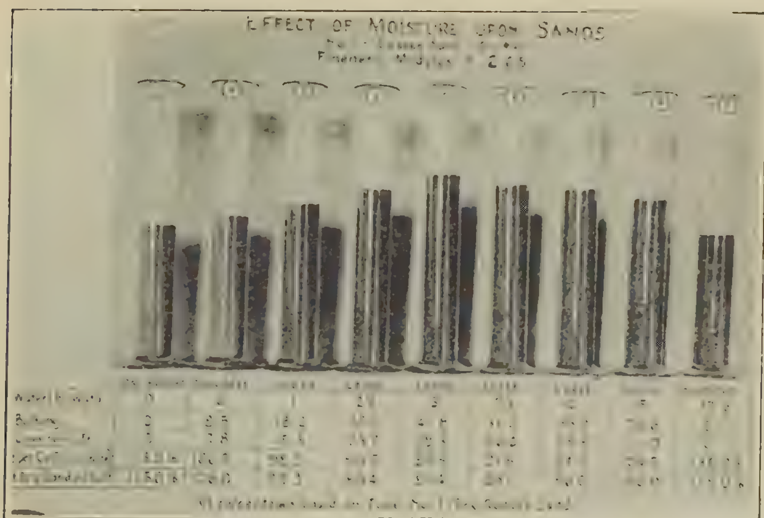
The method of designing concrete as described in this paper appears at first very involved, while, as a matter of fact, once the basic principles are understood there is no reason why this method cannot be applied on any work regardless of its size.

I cannot impress too strongly on you to whom the design of concrete mixtures is new the absolute necessity of adopting standard methods for making all your tests and determinations as set forth by the American Society for Testing Materials.

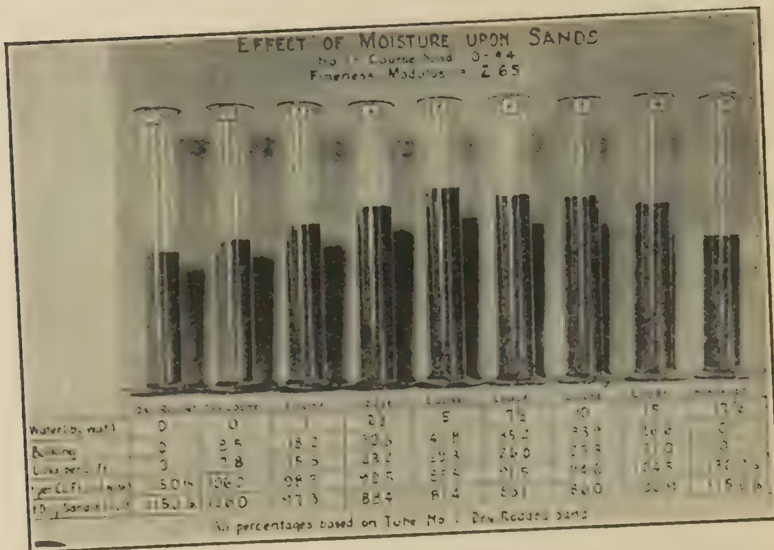
Every factor in the making and curing of test specimens has a direct bearing on the relative results or values of the tests.

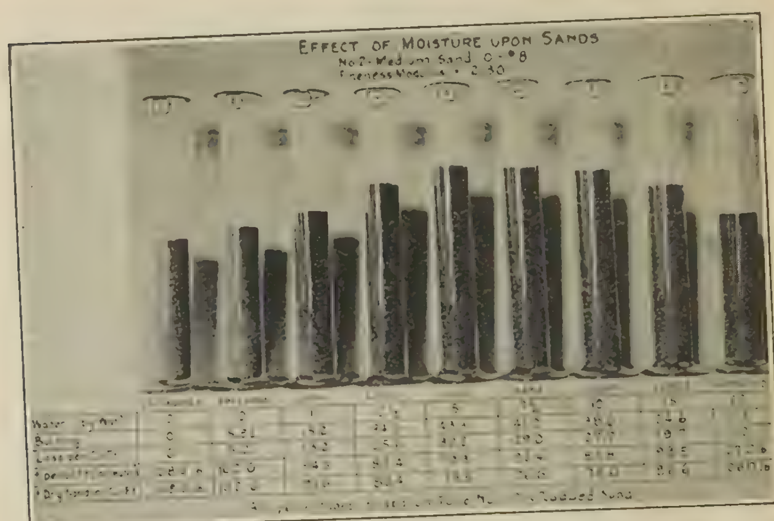
All test cylinders should be made at the mixing plant under as nearly uniform practice as is possible, as it is usually very difficult to get representative samples from the forms. If specimens are taken from the forms they should be taken from batches from which comparative specimens have been taken at the mixing plant.





GRAPHICAL EXAMPLES OF THE BULKING OF DIFFERENT SAND CONTAINING VARIOUS AMOUNT OF MOISTURE.





GRAPHICAL EXAMPLES OF THE BULKING OF DIFFERENT SAND CONTAINING VARIOUS AMOUNT OF MOISTURE.

Test cylinders should be made at a point where they will remain untouched for forty-eight hours. Our experience in the matter of curing test cylinders is a good example of the lack of taking proper precautions to insure uniform results. As stated before, we made our test cylinders as prescribed, cured them in the air 48 hours, buried them for 24 days and sent them to a local laboratory to be crushed at the expiration of 28 days. We obtained remarkably uniform results from the cylinders made in the warm weather and buried in damp sand under a field office.

When the weather became colder we buried the test cylinders in a shallow cellar. Too late we found the temperature in the cellar approached the outside variations. We hastily installed a heating system which experience has taught us was very defective. Cylinders cured in the cellar without heat were uniformly lower in strength, while the specimens cured in the heated cellar were erratically higher in strength. The gas heater installed was placed in the center of the cellar and as time went on the heat dried out the earth in the cellar and the cylinders when tested gave different results traceable to the varying conditions in the curing. Those near the outside walls gave lower strengths than those nearer the heater. Some of the latter when uncovered were comparatively dry on the upper side and moist on the bottom.

Some of the test cylinders of concrete placed in cold weather were cured the first 48 hours in the corner of a frame boiler house. An examination of these specimens disclosed that the side nearest the boiler dried faster than the opposite side and the crushing of these cylinders invariably developed a vertical fracture down the center of the specimen.

The point that I am trying to bring out is there should be a standard practice for the carrying on of field tests which would be specific and practical for any situation and especially as it affects curing of test specimens.

Some device or method for the curing of specimens the first 48 hours at a constant temperature, summer or winter, should be developed, and the curing for the longer period in damp sand could be carried out under the same conditions at a very slight expense, which would insure field test cylinders of some value as compared with the haphazard practices which obtain as a general rule on the average construction job where test cylinders are being made.

DISCUSSION.

Mr. Nichols. CHARLES E. NICHOLS.—You speak of using accurate measurements for aggregates; did you use commercial batchers of sand and stone or something you designed yourself?

Mr. Watson. MR. WATSON.—We used Blaw-Knox batchers.

Mr. Foster. C. B. FOSTER.—Were any tests made as to bridging or arching of concrete around the rods? I have found that concrete will bridge and leave a void under the rod and not make a good job.

Mr. Watson. MR. WATSON.—We are perfectly satisfied that we had nothing like that happen. The concrete girders of this structure were heavily reinforced. The bottom center section had four layers of eleven 1¼-in. square twisted rods spaced on 3-in. centers in the vertical plane and the rods spaced 4-in. centers in the transverse plane.

Before we commenced pouring concrete we placed a batch of mortar, of the same consistency as the mortar in the concrete, to a depth of 1 in. on the bottom of the girder for a distance of about 10 ft. We then commenced placing the concrete from one end of the girder to a depth of 2 ft., of such a consistency that the weight of the concrete displaced the mortar previously placed and rolled it ahead of the concrete.

The concrete was of such a consistency that by puddling with paddles we had a homogeneous mass that flowed between the rods. We paid particular attention to the inspection of this operation to avoid bridging and voids so that in the construction of sixty-five 40-ft. girders we did not have 3 sq. ft. of honeycomb on the entire surface of the girders and at the points honeycombed the depth of voids was negligible.

Mr. Howe. HENRY L. HOWE.—I would like to ask whether you made any comparisons between the laboratory loose method of measuring material and the results that are actually obtained with the Blaw-Knox batchers?

Mr. Watson. MR. WATSON.—That is a good point. In making all these determinations there is a certain amount of common sense and practical knowledge required. Some might think that the measurement of a unit of loose sand by volume regardless of conditions would give you the same net volume. This is not true as the density of the sand in the measuring device will vary according to the method employed for charging the measuring device. The density of sand measured loose in a wheelbarrow will vary considerably from sand measured in a batcher charged from a hopper, or a different density of sand would be obtained to some extent from whatever different method was employed. We approached as near as we could in making our field determinations the fall of the sand in the hopper.

In other words in making our bulking determinations we dropped the sand into our receptacle from a fixed height estimated to be the average

mixer and separating it back into its original parts of sand and gravel and found that the proportion of sand was practically the same as our designed volume.

Mr. Slater.

W. A. SLATER.—The information that has been given here brings out the importance of a fact we observed in connection with the tests to check up the strength given in the Joint Committee Report tables. These tests were made at the Bureau of Standards last fall, on concrete of 72 mixes selected from the Joint Committee tables. On plotting the strength against the water-cement ratio, we found that the curve, $\dfrac{14{,}000}{7^{\frac{w}{c}}}$, fitted the average of the points fairly well, but that the spread above and below fall of the sand from the hopper into the batcher so that our test would approach the same density as the sand in the batcher. We checked this on several occasions by taking a unit of freshly-mixed concrete from the the curve was very much more than desirable. In fact there seemed to be two or three distinct curves, depending on certain properties of the aggregates. After searching for some time, someone suggested that we had harsh mixes, and following that clue, we cut out all the points in which the sand was less than one-third of the total aggregate. We found that when we cut out those points all the rest except two or three fell on or above the water-cement ratio curve, and that came to my mind in connection with the following of these mixes. I noticed that in the mix arrived at, the sand was considerably less than one-third of the total aggregate, but on account of the change in the mix, you finally arrive at a mix which had more than half of the sand. I do not want to speak of what I have said here as perfectly general, because I suppose it is too much to expect this to be of general application without fail, but I think it is a precaution probably worth observing, that the sand ought to be equal to or greater than one-third the total aggregate.

Mr. Watson.

MR. WATSON.—In placing the 15,000 cu. yd. of concrete in this structure we changed the proportions of the batch for each car of sand and gravel used.

We did not attempt to use specified graded aggregates until we found by experiment which of the aggregates were most suitable for the various parts of the structure. The mixtures ran in very strange proportions. Upon one occasion we were unable to mix a full batch as the designed mix had such a large proportion of sand that we could not measure the volume of gravel required because the gravel batcher was not designed to measure a volume so small. This only applied when we were using an extremely coarse sand and an unusually coarse gravel. This sand was so coarse that it was not economical or desirable to use it and the remaining cars on hand were not used for concrete aggregate.

The designing of the mixtures for this work disclosed that we did not

have ten mixtures of loose materials that the proportions were actually identical. Some were very close and I do not mean to infer that it is impossible or impractical to use a fixed proportion under certain conditions but we were proving out the theory of the designed mix and were trying to satisfy ourselves to determine the merits of the method using the various sized aggregates and the results of our test cylinders are so close to the estimated strengths that we are satisfied the slight additional effort required in the field to make the necessary determinations is justified by the excellent results.

Mr. Dunnells. C. G. DUNNELLS.—Has the Pennsylvania R. R. any decided preference for square twisted bars over other forms of deformed bars?

Mr. Watson. MR. WATSON.—Yes. In our opinion it is an excellent test of each individual rod. If you twist a square rod cold you are quite sure you have a sound rod. The twisting also removes the shop scale, which is very desirable.

Mr. Douglas. A. S. DOUGLAS.—How about the commercial aspect of this subject? In a suit at law they sometimes introduce the element of mental anguish. I imagine you had some mental anguish. I wonder if it was compensated in the case of cement by a lower safety factor or any other way.

Mr. Watson. MR. WATSON.—We designed our concrete mixtures in the girders for 3,000 lb. strength. We knew that these members were designed for an extreme fiber stress of 675 lb. for concrete in compression so that we were not taking any chances.

We are going to build a similar structure this year and have decided to design the concrete for 2,500 lb. in all the reinforced concrete, 2,000 lb. for gravity sections exposed to the weather and 1,500 lb. in the foundations the tops of which are 3 ft. below the ground level. I would recommend for any concrete work exposed to the weather a minimum 28 days strength of 2,000 lb.

Unless a detailed specified method of making field tests is embodied and adhered to the specification for a definite strength concrete is one that is very likely to cause much mental anguish and considerable legal difficulty in case of a dispute.

Regarding economies, by designing our mixtures, controlling the water-cement ratio, and specifying definite minimum and maximum sized aggregates we were able to effect savings in the amount of cement used and in the cost of the aggregates as compared with the usual method of using arbitrary proportions regardless of the sizes of the aggregates.

Mr. Lindstrom. ROBERT S. LINDSTROM.—I understand that you use the colorimetric test on sand. Did you use any other test to verify the colorimetric test? What did you find in the colorimetric test?

Mr. Watson. MR. WATSON.—In the Pittsburgh district we have an arbitrary unwritten specification only permitting the use of sand from certain portions

of the rivers from which the aggregate is obtained. We have found the colorimetric test to be very valuable in detecting the presence of soft coal. Any percentage of soft coal in the sand is almost instantly revealed by the colorimetric test.

The sand we used was free from coal and it was not necessary to condemn any sand received because of other organic impurities.

Mr. Lindstrom.—The reason I asked that question was because I have had a number of tests on sand and have found what you say is true—it brings out the coal dust. **Mr. Lindstrom.**

Mr. Watson.—Yes, and if there is any foreign matter in the sand the colorimetric test is a much better procedure than the usual method of taking a handful of sand and running it through your fingers and thereby making a decision as to its qualities. **Mr. Watson.**

Arthur A. Levison.—If permitted, I would like to go back to a previous question with regard to the Blaw-Knox batchers used on Mr. Watson's work. The question was asked how it is possible to determine what is going on inside the batcher. The U. S. Bureau of Roads in 1924 was interested in the same question. With an outfit of Blaw-Knox batchers they found out just what was going on inside the batcher with loose moist sand. The scheme involved considerable work. It consisted of setting the batcher at any given volumetric capacity, and filling the storage bin with perfectly dry sand; then filling the batcher with that dry sand and determining by weight the number of cubic feet of dry sand that the batcher contained. Following that the bin was filled with sand containing various percentages of moisture and the batcher was filled with the sand in those various moisture conditions, and by weight it was then possible, deducting the percentage of moisture, to ascertain the actual amount of dry sand contained in each cubic foot of loose, moist sand which the batcher automatically measured. **Mr. Levison.**

F. M. McCullough.—What was your experience with coarse sand? When it is necessary to heat the sand to thaw the frozen particles, does that introduce difficulty in bulking effect? **Mr. McCullough.**

Mr. Watson.—In regard to the first question; the coarse sands we used were entirely unfitted for reinforced-concrete work. Using the theory of the "Design of Concrete Mixtures," we obtained the desired strengths but not the required workability. Also it was much more economical in Pittsburgh to design mixtures using the finer sands as less sand is required and its cost being relatively high as compared to the cost of the gravel a saving to the contractor was made possible. **Mr. Watson.**

In answer to the second question, we were unable to detect any marked difference in the bulking of the sand caused by the injection of quantities of live steam into the hoppers before charging the batchers. In the tests we made, the moisture content seemed to remain about the same. Evi-

dently the portion of the live steam which condensed was offset by the vapor that arose from the hoppers.

Mr. Wilson. E. B. WILSON.—How often were those tests made? What was the effect of rain on your cars and on storage?

Mr. Watson. MR. WATSON.—We made these tests as soon as the cars came into the yard and our experience has been that the moisture content varied very slightly between the time of their receipt and the time they were used unless it rained. When it rained we had to make re-determinations and in the working of the job this did not cause the contractor any delay.

We plotted a curve of the moisture content and relative bulking of the sand of approximately 100 determinations so that by determining the moisture in the sand the relative bulking was taken from this curve without making a field determination of the relation of the volume of loose and rodded sand.

Was it your idea that the determination of the moisture content in the sand would take too long?

Mr. Wilson. MR. WILSON.—Oh no, not at all.

Mr. Watson. MR. WATSON.—A simple method, applicable to the sand we used, to determine the bulking of loose sand to a unit of sand dry and rodded is to fill a metal receptacle, approximately 10 in. in diameter and 11 in. high, level full with the sand to be used. Then remove the sand so measured and place in a bucket or other container so that the same volume of sand can be reused in the determination. Next, partially fill the metal receptacle with water and pour the sand of the sample back into the receptacle in three estimated layers and rod each layer 15 times with a ¾-in. rod and the resultant volume of saturated rodded sand is approximately the same as if it were dry and rodded.

By measuring from the top of the receptacle to the top of the flooded and rodded sand you obtain the bulking in inches. Dividing this measurement by the height of the flooded and rodded sand you obtain the percentage of bulking of the loose sand to a unit dry and rodded.

In presenting this paper on designing mixtures I intentionally detailed the various theoretical operations to impress on those, who are to be responsible for the results obtained, the necessity of study and some effort if you are to obtain the best results.

There unfortunately are many who, for some unexplainable reason, seem to think that the proportioning of concrete and its use should be made so simple that it should not require any special knowledge to construct satisfactory and lasting concrete work.

It is the prevalence of this idea that is responsible for the unsatisfactory concrete work throughout the country.

Mr. Turner. C. A. P. TURNER.—I question the value of the slump test on the ground that it is indefinite unless the temperature of the materials is constant.

This statement is based on experience where we had stone exposed to the sun on a long summer day and the heat in this aggregate, notwithstanding a liberal amount of water in the mix, set the concrete up before it could be dumped from the wheelbarrow 100 ft. from the mixer without chopping it out. In hot weather, in Omaha, a contractor had trouble with shrinkage and opening up of the splices in the slabs and could not understand the cause of the trouble. He was advised to cool the aggregate by wetting in advance of mixing, taking the excess heat out so that the cement would set normally and the trouble was thus remedied. In ordinary work the rating of the slump test for plasticity is of little practical value as an index of the workability unless the temperature of the sand, stone and cement is constant instead of variable as found on the job.

Field Control of Concrete on the Delaware River Bridge.

By A. W. Munsell.*

The purpose of this paper is to set forth the fact that even with specifications fixing the proportions for each grade of concrete, it is still possible to use the Abrams' theory, subject to the limits of your specifications, and thereby secure better concrete. As an illustration I shall use the concrete work on the Delaware River bridge now in course of construction between Philadelphia, Pa., and Camden, N. J. This is an especially good illustration because, after some unsatisfactory results, both in the appearance of the concrete and the test results, the Abrams' theory of proportioning concrete was partially adopted, and is constantly demonstrating its soundness and practicability.

General Features of Bridge.—The bridge when completed will be the largest bridge of the suspension type in the world, the span between piers is 1,750 ft. For 800 ft. of this span the clearance above mean high water is 135 ft.

The whole structure, extending from Sixth and Race Sts., Philadelphia, to Sixth and Penn Sts., Camden, will be 1.81 miles long. The roadway, 57 ft. wide, will accommodate six lines of vehicular traffic—three lines in each direction—with a capacity of 6,000 vehicles per hour. In addition, there will be two lines of surface car tracks and two lines for high-speed cars, making the bridge 125 ft. wide overall. Pedestrians will be accommodated by two overhead sidewalks, 10 ft. wide on the river span and 16 ft. wide on the approaches.

When completed, the bridge will require a total of 320,000 cu. yd. of concrete, 50,000 tons of steel and will cost $37,210,000, approximately divided into $26,210,000 cost of construction and $11,000,000 for real estate condemnation.

Historical Data.—By agreement, the Commonwealth of Pennsylvania and the city of Philadelphia divide the cost of the land and approaches on the west side of the river and one-half of the bridge proper, while the state of New Jersey is required to provide the land and approaches in Camden and one-half the cost of the bridge.

The joint commission was organized Dec. 12, 1919, at a meeting in Philadelphia, called by Governor Sproul, of Pennsylvania. Under date of Sept. 24, 1920, a board of engineers was appointed, composed of Ralph Modjeski, chief engineer, George S. Webster and Laurence A. Ball, to consider sites and plans for a bridge. A report was submitted by the board

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*Assistant Engineer, Field Division, Delaware River Bridge Joint Commission.

of engineers on June 9, 1921, and public hearings were held the following week by the joint commission. The joint commission approved the plans June 23, 1921. The plans for the bridge were approved by the Chief of Engineers, U. S. A., and the Secretary of War on Sept. 29, 1921.

Ceremonies were held Jan. 6, 1922, marking the beginning of construction of the bridge. The setting of granite on the two main piers was completed in Philadelphia and Camden in March and May, 1923, respectively. The Camden caisson had been sealed about thirteen weeks later than the Philadelphia one, the sinking having been started almost three months later than on the Philadelphia side, and the caisson being sunk about 24 ft. deeper in order to reach satisfactory rock.

Piers, Anchorages and Towers.—The piers of concrete, faced with Georgia granite, on which the towers are supported, are 70 ft. wide by 143

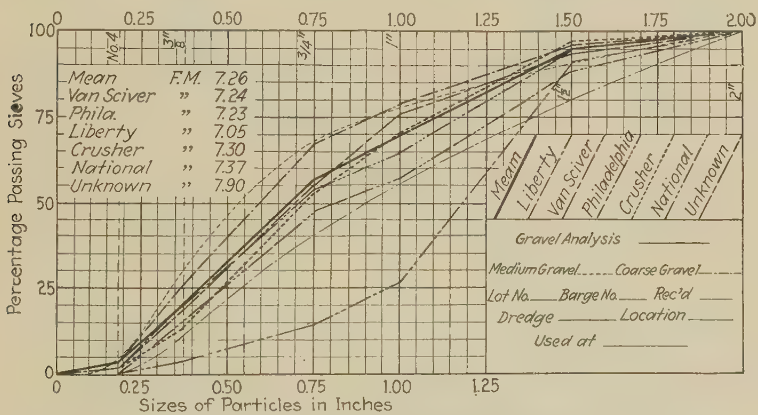


FIG. 1.—GRAVEL ANALYSIS CURVES.

ft. long and go down to rock which was found at El.-62 ft. on the Philadelphia side and El.-86 ft. on the Camden side. The Camden pier, with 24,190 cu. yd. of concrete, and the Philadelphia pier, with 31,300 cu. yd., were completed in March and May, 1923, respectively.

The anchorages have an overall width of 176 ft. and length of 245 ft., and have respective heights from rock foundation to top of 247 ft. in the case of the Philadelphia anchorage and 288 ft. in the case of the Camden anchorage. The first sections of the anchorages were constructed on two rectangular caissons 40 ft. by 125 ft. in section and eight 20-ft. diameter circular caissons which were founded at about El.-62 on the Philadelphia side. The same size circular caissons were used on the Camden side with rectangular caissons which were 40 by 140 ft. The Camden caissons were founded at about El.-105. All the caissons were sunk by the open dredging process using orange peel buckets through well-holes left in the caissons. There was placed 62,000 cu. yd. of concrete in the Philadelphia

anchorage and 85,300 cu. yd. in the Camden anchorage; these were completed in January, 1924, and March, 1924, respectively. In this portion of the structure the steel girders and eye-bar chains are embedded for anchoring the cables.

The main towers which support the cables are of slender outline and are designed to deflect with changes in pull of the cables. They rise to a height of 384 ft. above mean high water and were completed in May, 1924.

Cables and Suspenders.—Two cables with a completed diameter of 30 in. will support the floor system. Each cable is built of 61 strands of

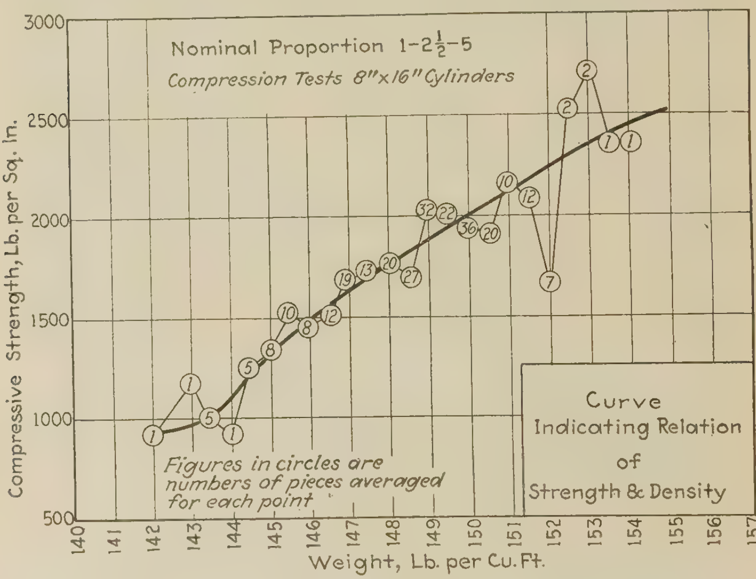


FIG. 2.—RELATION OF STRENGTH TO DENSITY.

306 wires each, making a total of 18,666 wires per cable. The diameter of wire used is 0.2 of an inch, which is not quite as thick as an ordinary lead pencil. Each cable is 3,550 ft. long and weighs 3,550 tons or one ton per foot. The total length of wire used was 25,100 miles. It took approximately five months to string the cables. The last wire was placed Jan. 23, 1925.

Bands for the suspender ropes are now being clamped on the cables and the suspender ropes, deck girders and stiffening trusses will be completed during the coming summer. Contracts have been awarded and work is in progress on both the Camden and Philadelphia approaches.

Aggregate.—The aggregates used on the job were supplied by the DeFrain Sand Co., whose source of supply is in the Delaware River above Bordentown, New Jersey, where dredges are used to raise the material

from the bottom of the river and separate it into commercial sizes. The large aggregate is usually round pebbles, but in the winter, when the dredges are pulled to shore on account of ice conditions, we are supplied with crushed pebbles. The maximum size of the large aggregate is 1½ in., which is the commercial size in the Philadelphia district.

Concrete Work.—The concrete plants were similar on both sides of the river and consisted of a battery of two 1-yd. Koehring mixers, operated by steam, equipped with Eastman timers for assuring one minute mixing

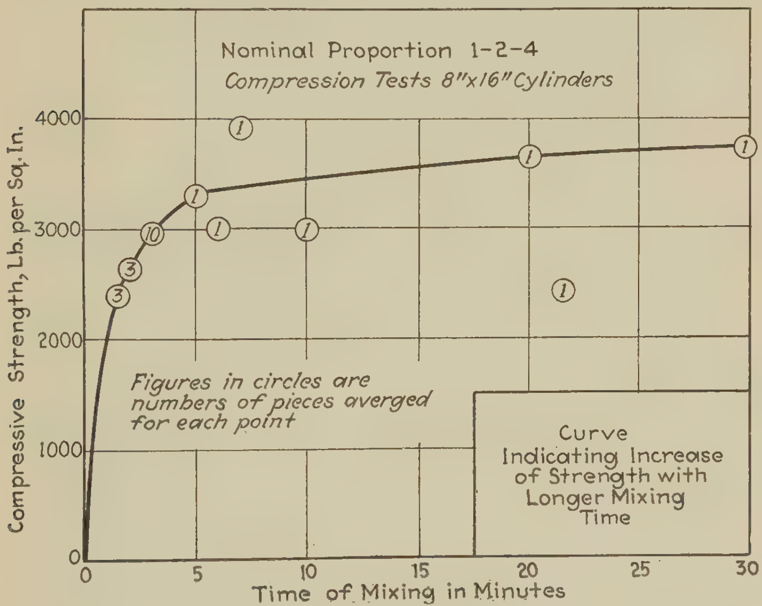


FIG. 3.—RELATION BETWEEN STRENGH AND MIXING TIME.

of the concrete. Overhead bins with a capacity of 120 yd. of sand and gravel were supplied by two stiff-leg derricks, operating three-quarter yard clam shell buckets, which transferred the material from barges placed alongside the plant to the bins. The aggregates from the overhead bins fed into open-top batch boxes, divided in sand and gravel compartments, on the sides of which were painted red marks for 1 : 2 : 4 proportions and green marks for 1: 2.5: 5 proportions. Any changes in the proportion for either grade of concrete were made by measuring from the red or green marks. The batches were struck down level in all cases with a short-handled hoe.

We are now using a much better batcher which will be described later. The batches of aggregates were dropped into a hopper behind the mixer, where the cement was added. Water was added from an ordinary

water barrel with a gauge attached to the side for fixing the quantity. From the mixer the concrete was chuted into an elevator skip and raised about 40 ft. to another hopper from which it was dropped into 1-yd. bottom-dump buckets on an industrial railway, which transferred it to where it could be reached by a boom from a locomotive crane or stiff-leg derrick.

During cold weather protection was given all new concrete by covering with tarpaulins kept about 6 in. above the concrete. In this space steam pipes were led, in other parts of the work salamanders were used. As a minimum temperature of 50 deg. F. was specified for concrete setting in cold weather, maximum and minimum thermometers were used to record the actual temperature for the protected concrete. In other places 1-in. diameter pipes were left in the concrete in which thermometers were suspended in kerosene to observe the temperature changes over a period of time.

Testing Methods.—Inspection of the aggregates and testing of the concrete was carried on by a force of three inspectors, one man on each side of the river, taking samples of aggregates and making test pieces and slump tests for control of water, and one man in the testing laboratory of the city of Philadelphia, who looks after the breaking of the test pieces.

The inspectors' duties consist of taking samples of the barges of sand or pebbles as received. A mechanical analysis is made of each sample and plotted upon our aggregate curves which serve a double purpose, that is, to determine if the aggregates meet our specification and also determine their fineness modulus. The proportions are fixed by the fineness modulus theory, as described in Bulletin No. 1 of the Structural Materials Research Laboratory, entitled "Design of Concrete Mixtures" by Prof. Duff A. Abrams.

Each side of the river is furnished with the following equipment, viz.: a complete set of sand screens ranging from No. 4 to No. 100, and gravel screens of ¼-in., ⅜-in., ¾-in., 1-in., 1½-in. square mesh standard, with a Fairbanks double-beam scale with a capacity of 300 lb. reading to ¼ oz. Three 8-in. diameter by 16 in. high cylinder molds are on hand at each mixing plant for making test pieces. A slump cone with dimensions of 5 1/3-in. top, 16-in. height and 10 2/3-in. base is used for making consistency tests immediately before a test piece is made, and for control of water at any time the inspector thinks the concrete is getting too wet. A test piece is made for each 100 cu. yd. of concrete during the day, or for each run of concrete when the proportions are changed. Samples of concrete for testing are usually taken at the mixer, but are occasionally taken from the mass.

The method for taking the sample is for the inspector to stand on one side of the chute with a shovel, as the mixer is discharged, shovel fulls are taken of the run which are deposited in two 12-qt. galvanized-iron water buckets. The buckets are dumped onto a ⅜-in. thick steel plate, which has been set level. The concrete is remixed with a shovel and a slump test made for consistency. After the slump is taken the concrete

is remixed and a test cylinder made. The test cylinders are broken at 28 days, but when more than two cylinders are made, in the same proportion, the extra cylinders are extended to 60 days, 6 months or one year.

Colorimetric tests are made on all samples of sand, and barges showing a color darker than No. 2 in the scale are rejected.

All cement is sampled at the mill and tests are made before shipment.

The methods used for making test pieces and slump tests are those recommended by the A. S. T. M. and are as follows: For the slump test,

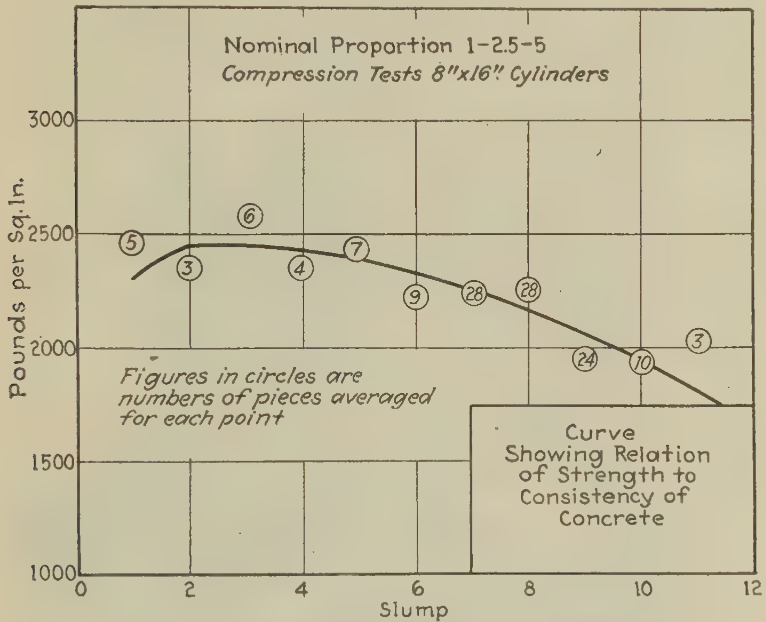


FIG. 4.—RELATION BETWEEN STRENGTH AND CONSISTENCY.

the cone is filled with concrete to 4 in. of its height and tamped with a ¾-in. round steel bar, 20 in. long and with a bullet-shaped point, for 30 strokes. The cone is filled another 4 in. and tamped 30 strokes, having the tamping rod penetrate through the second 4 in. into the first and bind the two layers. The operation is continued in 4-in. layers until the full height of 16 in. is reached. The top is then struck off and the cone lifted vertically, letting the mass slip out gently. The cone is set alongside the mass, the bar placed across the top and the height taken from the under side of the bar to the top of the mass, which is called the slump in inches.

An 8 in. by 16-in. cylinder mold with a bottom is placed on a level space and the same operation in making a slump test is used for making a test piece. The top is struck off and as soon as the concrete is partially set the identifying number is scratched in the top with a nail. The test

pieces are allowed to set for 48 hours in a warm atmosphere, although the forms are loosened at the end of 24 hours they are then taken to the field office and placed in damp sand storage until the time arrives to break them.

Fig. 5 is a gravel shaker which helps in the rapid determination of the mechanical analysis of a sample of gravel. In operation, a weighed

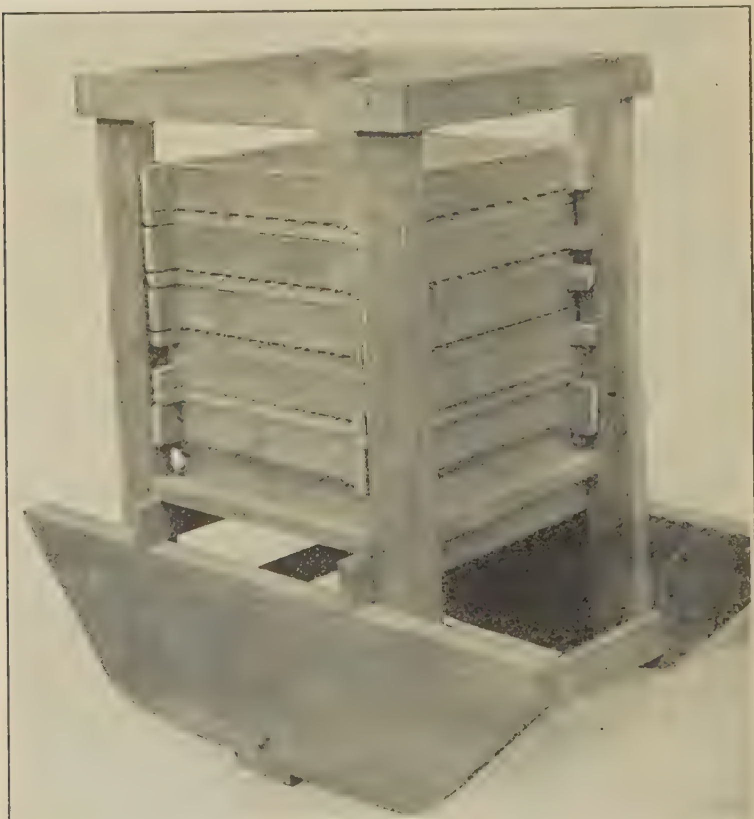


FIG. 5.—TESTING DEPARTMENT'S GRAVEL SIEVES IN SHAKER.

sample of 25 lb. is dumped on the top screen and the frame shaken until no more pebbles come through. The set of screens in order from the top are the 1½ in., 1 in., ¾ in., ⅜ in., and ¼ in., with a galvanized iron pan at the bottom to catch the material coming through the last screen. The screens were all of the Tyler standard. Each piece of screen is 20 in. square outside and the frames are of ¾ in. by 4 in. high pine strips

mortised at the corners. The screens are held on the frames by ¾-in. square strips screwed to the screen frames. The shaker itself is made of 2 in. x 12 in. x 3 ft. 6 in. rockers with 2 in. x 4-in. standards braced with ¾ in. x 4-in. pine strips screwed to the main frame. A ¾-in. x 4-in. strip

METHOD FOR DETERMINATION OF CORRECT PROPORTIONS
BY THE ABRAMS THEORY

Nominal proportion = 1-2.5-5
Total unmixed aggregate = 7.5 cu. ft.
Total mixed aggregate = 7/8 x 7.5 = 6.56 cu. ft.
True mix = 1-6.56

Mix. Cem-Agg.	F.M. required for Agg. 0-1½" (Bulletin #1. Table #3)
1-7	5.55
1-6	5.65
By interpolation	
1-6.56	5.61

Sand Analysis
Weight of sample taken 500 gm.
Lot No. 1143

	gm.	%	F.M.
Held on #4	23	4.6	4.6
8	55	11.0	15.6
16	58	11.6	27.2
30	126	25.2	52.4
50	199	39.8	92.2
100	35	7.0	99.2
Passing 100	4	.8	291.2
	500	100.0	

F.M. = 2.91

Gravel Analysis
Weight of sample taken 25 lbs.
Lot No. 1144

	lbs.	%	F.M.
Held on 1½"	1.5	6	6
1	7.5	30	~~36~~
¾	3.0	12	48
⅜	8.5	34	82
#4	3.5	14	96
Passing #4	1.0	4	500
	25.0	100	732

F.M. = 7.32

Formula for proportioning of aggregates. (Bulletin #1)

$$p = 100\,\frac{A-B}{A-C} \qquad p = 100\,\frac{7.32-5.61}{7.32-2.91} = 39\%$$

where p = percentage of fine aggregate in corrected proportion.
A = fineness modulus of coarse aggregate. (7.32)
B = fineness modulus of total aggregate. (5.61)
C = fineness modulus of fine aggregate. (2.91)

39% of total aggregate (7.5 cu. ft.) = 2.9 cu. ft. or fine aggregate in corrected proportion.
Corrected proportion = 1-2.9-4.6

Cylinders made with corrected proportions for Lot Nos. 1143 and 1144.

Slump (16" cone)	Proportions	Weight per cu. ft.	Pounds per sq. in.	Calculated pounds per sq. in.	% of Actual pounds per sq. in.
3"	1-2.9-4.6	152.2	2490	2325	107.1%
2"	1-2.9-4.6	153.8	2555	2387	107.0%

FIG. 6.—PROPORTION DETERMINATION BY ABRAMS' METHOD.

is also fastened across the bottom of the rockers so the material will be jarred each time the shaker is pushed from side to side.

The Abrams' theory of proportioning concrete was partially adopted about two years and a half ago after some unsatisfactory results both

in the appearance of the concrete and the test results. The use of the theory was limited in our case by the specifications fixing nominal proportions of 1: 2: 4, 1: 2.5: 5 or 1: 3: 6, but it allowed changes in the total proportions of 1: 6, 1: 7.5 or 1: 9, also it was ruled that the proportion of sand should not be less than the nominal proportion for each grade of concrete. The thought was that cutting the sand would make the concrete too harsh. We could, however, raise the sand content whenever analyses indicated its necessity. These restrictions were made in the concrete used on the first sections of the anchorages, but the results obtained, even with this handicap, were so satisfactory that they were lifted and the contracts now in force only restrict to the total of proportions for each grade of concrete.

The later results are all from concrete placed in the first sections of the Philadelphia and Camden anchorages and are usually of the 1: 2.5: 5 proportion, because there was more of this grade of concrete placed and of course more test pieces made. The description of plant and handling of concrete also applies to this portion of the anchorage.

On later contracts central mixing plants were used, equipped with either the Johnson or Blaw-Knox batchers giving a struck-off measurement. Both types have been satisfactory in service and are strongly recommended as economical for the contractor and do away with the "hit and miss" method of using the open-top batch box which is piled high on one batch and low on another as fixed by the eye of the man operating the gates. With the struck-off measure given by the above types of batchers the quantities determined by our mechanical analyses are assured.

Application of Abrams' Theory.—Fig. 6 is a tabulation of the results obtained using the methods outlined in Bulletin No. 1. The first step after the samples are taken is a screen analysis of both materials. The examples of sand and gravel analysis here shown are actual analyses taken at random from our records and given exactly as we make them. The first column under sand analysis is the sizes of sieves. The second column is the amount held on each sieve in grams. The third column shows the amounts held, converted to percentage of the total held. The fourth column is a conversion to fineness modulus, which is the sum of the percentages held on each sieve divided by 100. The gravel analysis is the same arrangement except 500 is added to the total held, because all this aggregate would be held on the sand sieves, and 100 is added for each sand sieve. Also the 1-in. size is struck out because in the theory it is called a half size and only whole sizes are considered.

Our proportions are fixed by the specifications to 1: 2.5: 5. Thus, according to Bulletin No. 1, p. 11, paragraph 4, if more than 15 per cent is coarser than any sieve the maximum size shall be taken as the larger sieve in the standard set. This gravel analysis shows that 48 per cent is held on the ¾-in. sieve, therefore this lot of gravel takes the next size—the 1½-in. sieve. The grading then becomes from 0 to 1½ in. Then, taking into consideration that in combining the aggregates the volume

shrinks about one-eighth, our nominal proportion of 1: 7.5, as fixed by the specifications, then becomes 1: 6.56, the true mix. The next step is to fix the proper fineness modulus for this proportion, which is found by interpolating from Table 3, Bulletin 1, the fineness modulus between 1: 7 and

Fig #6

FIELD CONTROL OF CONCRETE ON DELAWARE RIVER BRIDGE.

SPECIFICATION CONCRETE TESTS.
ANCHORAGES CONTS. NO.4 AND NO.5.

Compression Tests 8"X16" Cylinders
Summary

	Age Days	Prop. (Nominal)	Pounds per sq. in.	Wt. per cu. ft.	Remarks
Average	28	1-2-4	2731	148.9	(Spec require 2000 #/sq. in.
Minimum			1589	146.0	
Maximum			4145	153.3	
Mean variation from Average			13.5%	1.1%	

101 Pieces - 94.5% are above 2000 #/sq. in. (Spec. require 80% above 2000 #/sq. in.)
6 Pieces - 5.5% average 89.5% of 2000 #/sq. in. (Spec require remaining 20% to be above 75% of 2000 #/sq. in.
107 Pieces = Number Made

	Age Days	Prop. (Nominal)	Pounds per sq. in.	Wt. per cu. ft.	Remarks
Average	28	1-2.5-5	2066	148.3	(Spec. require 1500 #/sq. in.)
Minimum			1036	144.0	
Maximum			3379	152.0	
Mean variation from Average			14.6%	0.95%	

172 Pieces - 98.8% are above 1500 #/sq. in. (Spec. require 80% above 1500 #/sq. in.)
8 Pieces - 1.2% average 88.6% of 1500 #/sq. in. (Spec. require remaining 20% to be above 75% of 1500 #/sq. in.)
180 Pieces = Number Made.

	Age Days	Prop. (Nominal)	Pounds per sq. in.	Wt. per cu. ft.	Remarks
Average	28	1-3-6	1841	149.6	(Spec. require 1000 #/sq. in.)
Minimum			1290	147.2	
Maximum			3098	152.6	
Mean variation from Average			15.1%	0.87%	

37 Pieces - 100% are above 1000 #/sq. in (Spec. require 80% above 1000 #/sq. in.)

Extracted from report on contracts No. 4 and No. 5 on tests of concrete made in accordance with the Abrams theory.

FIG. 7.—SPECIFICATION TESTS ON ANCHORAGE CONCRETE.

1: 6, which is 5.61. Then, using the formula for finding the percentage of sand, the figure of 39 per cent is arrived at. With this percentage our proportion of sand becomes 2.9 and the gravel 4.6, or a 1: 2.9: 4.6, or 1: 7.5.

In this figure are also two tests for the concrete made using these aggregates. The calculated strength with the slumps used were found to be slightly less than the actual strengths.

The various steps at arriving at the right proportion as outlined above takes about one hour, which is principally used up in securing the sample. Our specifications require that barges be placed for sampling 24 hours before required but it is honored more in the breach than in the observance. The barge is usually placed at the mixing plant during the night and is found when we arrive for work. Our practice is to use the nominal proportion whenever a new barge of sand or gravel is placed. Testing time is lessened materially by the fact that tables of figures are prepared for a wide range of percentages of sand, and all that is required is to make the mechanical analysis of the aggregates and then read off the proper proportions.

With the adjustable batchers now in use sticks ½ in. square and about 18 in. long are prepared with markings in proportions, on one side for sand, and on the opposite side the complementary proportion for gravel. The sticks are held up to the sides of the telescoping portion of the batcher and either raised or lowered as indicated.

Test Results.—Fig 7 is a tabulation of the results obtained after the methods outlined in "Design of Concrete Mixtures" were adopted with the restrictions of our specifications and is extracted from a report made on these two contracts. The average strengths for all grades of concrete were well above the requirements of the specifications. In the case of the 1: 2: 4 nominal proportion it was about 37 per cent above the designed strength of 2,000 lb. for this grade. The 1: 2.5: 5 was about 38 per cent above the designed strength and the 1: 3: 6 nominal proportion was about 84 per cent above specification, but the high breaks of the last proportion of 1: 3: 6 are explained by the fact that this was mixed very dry. Most of this concrete had slumps of less than 4 in. We had one very wild break of 3000 lb. which we think belonged in another grade but we were never able to prove it.

Opportunity to compare our results with others using the same materials and mixtures was given in the report on Field Tests of Concrete Used in Construction, submitted to the Joint Committee on Standard Specifications for Concrete and Reinforced-Concrete by W. A. Slater and Stanton Walker, and published in the January, 1925, *Proceedings* of the American Society of Civil Engineers. The tests covered in this report were on the Victor Talking Machine Co.'s building, Camden, New Jersey. The aggregates used were from the same sources in both cases but they used a ¾-in. maximum size and we used a 1½-in. maximum. This difference in size of aggregate may account for the higher average strength that our results show, because, according to the theory, the larger the aggregate, with proper grading, the better the strengths that will be secured.

A study of our results gave us opportunity for various comparisons such as the relation of density to strength, consistency to strength, effect

	Victor Talking Machine Building Camden Tests	Camden Anchorage
Design strengths	2000 Lbs.	2000 Lbs.
Size of test piece	6″ x 12″ cyl.	8″ x 16″ cyl.
Average	2190	2731
Percentage of total pieces meeting required strengths	75% to 80%	94.5%
Total number of specimens	187	107
Percentage of total number of specimens showing strengths not greater than 80% of average.	10%	11.2%
Not less than 120% of average	10%	4.7%
80% to 120% of average	80	84.1
Not greater than 80% of design strength	5%	1.8%

of the time of mixing on strength, which are set forth in the following figures and in a general way confirms the results of other investigations.

Fig. 2 is a graph showing the relation of strength to density, as determined by the weight of the test pieces. The results shown are from a tabulation of 262 test pieces, 1: 2.5: 5 proportion used in the Camden and Philadelphia Anchorages (first section). Seventy-nine per cent of the pieces are between the weights of 145 and 150 lb. per cu. ft. This was a check on the uniformity of the making of the test pieces, and also indicates that the strength does increase with the density.

Fig. No. 3 is a curve indicating increase of strength with longer mixing. The results were taken as we could get them, when, from train or other delays a batch stayed in the mixer longer than the usual one minute. It represents 23 tests, 17 of which were made from batches with a mixing time between two and five minutes. The number shown is too few upon which to base any conclusion, but taken with the results of other investigations it certainly indicates the value of longer mixing for the shorter times up to five minutes.

Fig. No. 4 is a graph showing the relation of consistency, measured by the slump test, to strength. These results are from a tabulation from 127 test pieces of 1: 2.5: 5 nominal proportion from concrete placed in the Camden and Philadelphia anchorages (first edition) from proportions designed with the Abrams' theory. The strength required by our specifications for this grade of concrete is 1,500 lb. It shows that the strength increases with the dryer consistencies. It will be noted that this graph also shows that nine pieces of the 127 pieces had a slump of 7 to 10 in., a very wet consistency, but with this wet concrete we were still able to keep our strengths above the specifications. The figures in the circles are the number of test pieces which were averaged for the points on the

graph. The figure also shows the maximum strengths with a two or three-inch slump using the 16-in. cone which corresponds to ½ to 1-in. slump using the 12-in. cone, or our relative consistency is 1 with a two to three-inch slump using the larger cone.

The gravel analyses curves in Fig. 1 are from a number of samples taken to check the sampling of our barges. Lot No. 851 represents the results of sieve analysis plotted on our regular report form with an analysis of a sample of the same gravel which was obtained by washing a sample of concrete through a No. 4 screen with a hose and then putting the sample through the standard set of screens. This sample is shown on the graph as No. 851-A and shows a good check on our original analysis.

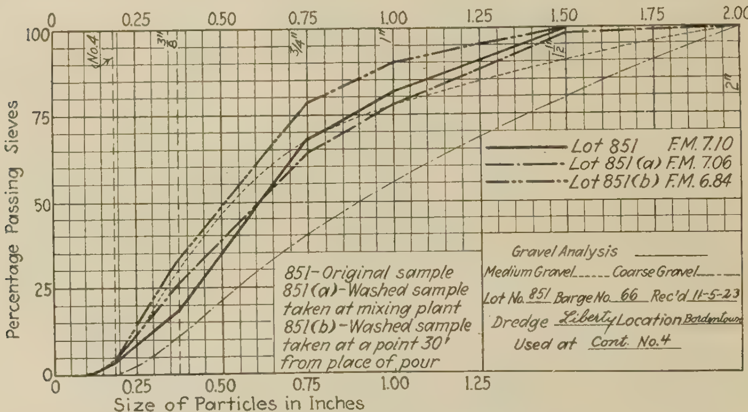


FIG. 8.—GRAVEL ANALYSIS WITH AVERAGE CURVE.

Still another sample of the same gravel was obtained by taking a sample of the concrete at a point about 30 ft. away from where it had been deposited and washing it through a No. 4 screen. This is shown as No. 851-B. The object of the last sample was to show that the concrete as it flowed or was pulled with a rake from the place of deposit was not carrying the larger aggregate because it was too wet, also that hoes and shovels should be used to spread the concrete to its final position. The change in the grading of the aggregates is also shown by the fineness modulus. The original sample of gravel had a fineness modulus of 7.10. The sample taken of the concrete at the mixer had a fineness modulus of 7.06, a difference of only 0.04, while the sample of concrete taken from the mass had a fineness modulus of 6.84, a difference of 0.26.

The characteristic break in the curve is to be noted in these three samples between the ¾-in. and the 1-in. size. We have noted this in all the samples taken for checking our sampling. The curves for these sizes in all the samples are very nearly parallel and we assume from that that our system of sampling aggregates is satisfactory.

There has been no difficulty with the two large material companies in the Philadelphia district in securing the grading of aggregates that we require. They have co-operated with us in every way, in fact they call us up to inquire how their material is running, and if it has not been satisfactory they have moved their dredges to new locations in order to give us what we want.

Figs. 8 and 9 are graphs showing the average curve for gravels and sands received from various sources. Both of the graphs are shown on our regular report form for gravel and sand. On the gravel form (Fig. 8) it will be noted that certain curves are drawn fixing the limits for a coarse gravel (the lower curve) and a medium gravel (the upper curve).

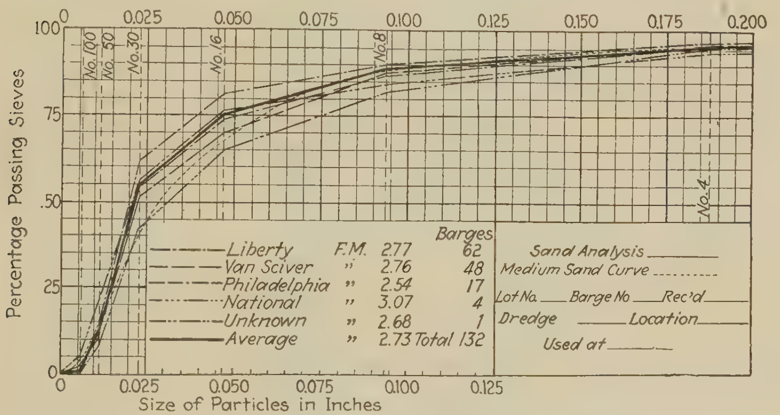


FIG. 9.—SAND ANALYSIS WITH AVERAGE CURVE.

The limits were fixed from the results obtained on our first contract and were fixed so that there would be a reasonable range in the sizes and near what could be expected under ordinary production methods. The graph shows that all the material received from the ordinary sources were well within the limits set except one which is shown on the graph as "unknown." As a matter of fact this material was from a stockpile in the yard of the material company and was of practically one size, held above the 1-in. screen. It was material coming from their crushing plant from which the ¾-in. material had been taken. It was so irregularly graded that it made wide changes in our proportion whenever, because of ice conditions in the river where the dredges were working, it became necessary to take this material. The difficulty was solved later by the mixing of one truckload of the large size with three truckloads of the ¾-in. size on the barges. The mixing was done by using a 1-yd. clam-shell bucket to lift and scatter the finer stone over the larger and was again remixed when it was lifted from the barge to the hopper over the mixing plant.

The fineness modulus of large aggregate ranged from 7.05 to 7.90 with a mean of 7.26 but the greatest amount of our material had an average range from 7.05 to 7.30, or a difference of 25 points.

The sand graph has a standard curve plotted on the form with which all samples are compared. This graph shows that our widest range was about 20 per cent at the No. 30 sieve but that the material of which we used the greatest amount had a range of only 10 per cent at this sieve.

Comparing the various sources by fineness modulus the range is from 2.54 to 3.07 with an average of 2.73, but the greatest amount of our materials had an average range of 2.54 to 2.77, or a difference of 21 points.

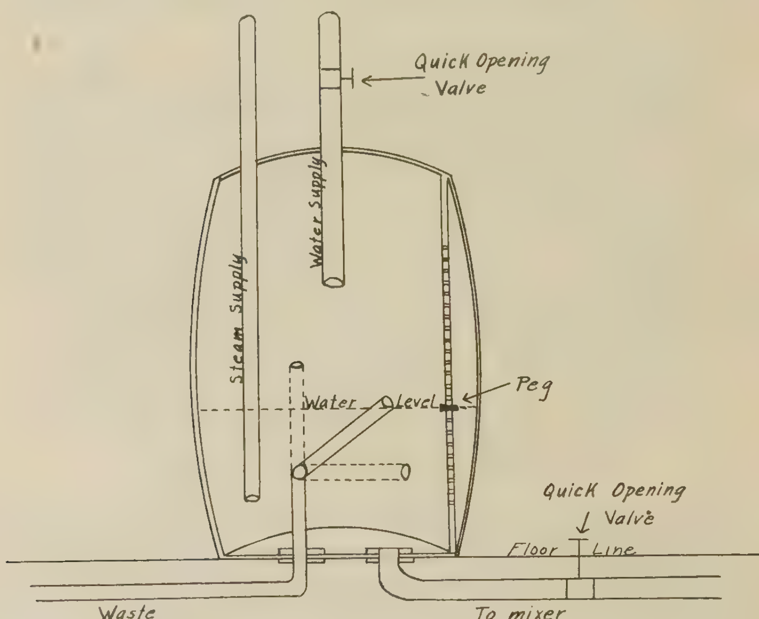


FIG. 10.—WATER BARREL USED TO CONTROL CONCRETE.

The results are from only the contracts using concrete on which we have complete results. On the other contracts, still under way, we are using better proportioning devices and better control of water.

Water Content.—This paper cannot be closed until something has been said about water. It has been almost every engineer's experience, until recently, that concrete wasn't considered good concrete unless it was wet—real wet. It was thought that the wetter the concrete was the easier it was to handle and the better the bond which was secured around the reinforcement rods. This is wrong. With a wet concrete, and by that I mean concrete with an 8-in. slump and wetter, you are continually chasing the accumulated water from one side of your form to the other,

and from corner to corner until at last you get it cornered, then you bail it out or drill a hole in the side of the form to let it out. With every deposit of this concrete, pushing the water ahead and from side to side, you are washing out the best of your cement, weakening your concrete and making laitance. The sooner we realize that good, plastic, workable concrete can be secured with a 4-inch slump or less, depending on the volume and grade of concrete you are placing, together with a proper time in the mixer and with men in the forms to push the concrete into place with their feet, we will begin to control the mixing water more closely thus insuring both a better grade of concrete and good surfaces with the forms removed.

Fig. 10 is a sketch showing the water barrel used for the control of water for concrete. An ordinary 50-gal. wooden barrel is used with a ½ in. x 2-in. wooden strip fastened to the side, perforated every ½ in. from a point 3 in. from the bottom of the barrel to the top, in which a peg is inserted for fixing the water to be used. The barrels are calibrated and the amount of water for each ½ in. of height is known. For rapid discharge a 2½-in. pipe line with a quick opening valve runs from the barrel to the drum of the mixer. For control of the maximum amount of water a 1½-in. pipe runs through the bottom near the side to a point 3 ft. from the bottom, where a series of two 1½-in. ells, connected by a short nipple, supports a pipe about 18 in. long. This movable arm is set on the peg at the side of the barrel and a greater amount of water than is shown by the peg is wasted. The device has been found practical in operation.

The active support of our board of engineers, composed of Ralph Modjeski, chief engineer, chairman; George S. Webster and Laurence A. Ball, with that of M. B. Case, the senior resident engineer directly in charge of the work, made the use of this method of proportioning our concrete possible, and as the efficiency of the method became apparent through the test reports and the appearance of the completed concrete work the restrictions, which were cautionary in the beginning, were removed and the methods adopted as standard as far as our specifications would permit.

Last, and by no means least, we had the intelligent and harmonious co-operation of the inspectors and engineers in charge of the placing of the concrete and control of water. To their efforts no small credit is due for the successful completion of this part of the job.

CONCLUSIONS:

From our experience we may draw the following conclusions:

(1) That the methods outlined in Bulletin No. 1, "Design of Concrete Mixtures," are practical for the field control of concrete.

(2) That while the fineness modulus theory appears involved its simplicity develops with use.

(3) That quite dry mixtures, as shown by the slump test, be used, and that two minutes in the mixer, after all materials are added, be required.

(4) That the use of batcher plants are economical for the contractor and insures the proper proportion for the engineer.

(5) That on any job of concreting, requiring the full time of the inspector, this method should be used for fixing the proportions with a proper control of water as measured by the slump test, regardless of whether strength test pieces are made.

(6) That a proper grading of aggregates, as shown by the mechanical analysis, is essential in any concrete work.

(7) That long chuting of concrete should not be permitted because the consistency, of necessity, must be very wet.

(8) That in cold weather, aggregates and water should be heated to a temperature above 50 deg. F. and the concrete in the mass be adequately protected with suitable heating devices, and an accurate knowledge of actual conditions be obtained by recording thermometers.

DISCUSSION.

C. E. NICHOLS.—Has any consideration been given, as these tests have shown such comparatively high strength compared with what was specified, to cutting down the proportion, even though the original contract specifications would not permit that? Mr. Nichols.

MR. MUNSELL.—Not yet; our board of engineers has not considered reducing them yet. Mr. Munsell.

H. F. FAULKNER.—How were the cylinders handled after having been made? Were they cured at a constant temperature with a constant degree of moisture? Mr. Faulkner.

MR. MUNSELL.—No; as I say, they were left standing for 48 hours with molds loosened after 24 hours, as soon as they had hard edges; that is, at 48 hours we took them over. In our field office we had a space in our sand box where we covered them with sand. We also kept a recording thermometer in there so that we knew when our temperature was getting lower than it should be, say below fifty, and put heat in there when we had to. We did not notice any changes in our cement from low temperatures, that is from storage. Mr. Munsell.

MR. WALKER.—I notice in the chart of Mr. Munsell's paper there is a difference in the compressive strength. I want to know in what way that can be accounted for? Mr. Walker.

MR. MUNSELL.—Didn't you use three-quarter-inch aggregate? Mr. Munsell.

MR. WALKER.—Three-quarter-inch aggregate. Mr. Walker.

MR. MUNSELL.—We used inch and a half. Mr. Munsell.

MR. WALKER.—I also did not notice Mr. Munsell mention taking into account the bulking of the sand in proportioning these mixes. Mr. Walker.

MR. MUNSELL.—We did not on those contracts. We are using it now. Mr. Munsell.

Notes on Laitance.

By R. M. Miller.*

The formation of laitance in the placing of concrete foundations under water, and in smaller measure in the pouring of concrete in the dry, has been regarded by many construction engineers as one of the inevitables—somewhat in the nature of death and taxes. The feeling has always seemed to be that it is something that we are to have with us always, the only remedy for which is to remove it after it has formed. Under the best of field conditions, it is literally the case that some laitance will accompany the pouring of concrete under water. It by no means follows, however, that we have no control over the amount, nor that its formation should be allowed to go to a point injurious to the concrete itself.

Description of Coal Pier Project.—The pouring of 3,900 cu. yd. of the seal concrete under water in the steel caissons of 48 foundations of the recently constructed Virginian Railway Coal Pier No. 2 at Sewalls Point, Va., developed extremely interesting results in the formation of laitance. Inasmuch as slime or laitance takes from the mix cement, particularly the finest particles from which the concrete derives its greatest cohesiveness, a study of the causes and a remedy is of value.

The bottom of the bay at the site of the Virginian Coal Pier No. 2 is of sand, clay and shells, overlaid with a blanket of mud or silt varying in depth from several inches to several feet. For each foundation into this bottom were driven 61 green foundation piles 40 ft. in length with cutoff at El.-27.75. Over these piles a steel form or caisson was placed and blocked firmly into position (Fig. 1). These caissons are 12 ft. in diameter at the top, 36.75 ft. high and flare to a bell shape at the bottom with a diameter at that point of 18 ft. After placing the cylinder the bottom was tested for mud. If indications were present of an amount more than two or three inches in depth it was pumped out.

Gravel was then spread over the bottom to a depth of approximately 6 in. The pouring of the 1: 2: 4 seal concrete was then begun with the use of a bottom drop bucket (Fig. 2) and carried continuously from El.-32.25 to approximately El.-20.0. The pouring of 90 cu. yd. of seal concrete consumed from six to seven hours during which time the cement-bearing water, within the caissons, was agitated almost continuously. The cement remained in suspension for some time after the completion of the work, the water clearing up, however, in from three to four hours.

*Formerly Resident Engineer, Virginian Terminal Ry., Sewalls Point, Va.

After the seal concrete had set from four to five days the water was pumped from the caisson and also the upper portion of the laitance which was more than 70 per cent water. The lower laitance was then mucked out to hard concrete. This concrete was then picked rough and thoroughly scrubbed before the concrete in the dry was poured continuously to the top. The laitance varied in thickness from several inches to several feet.

Materials and Methods.—The coarse aggregate used for the pier foundations was bank-run gravel of excellent quality, of good grading, from ¼ in. to 2 in., and clean. The fine aggregate used was a sharp coarse beach sand, testing 4 per cent silt and of practically no organic matter. The mix, 1 : 2 : 4, was carefully inspected at the plant. The time of mixing

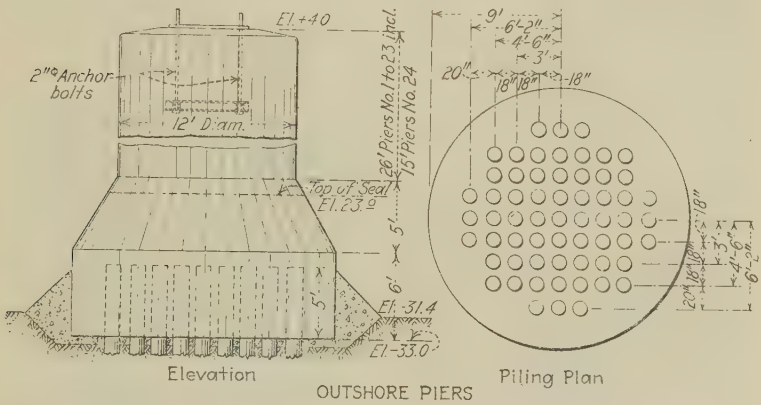


FIG. 1.—PLACEMENT OF CAISSON OVER FOUNDATION PILES.

was from 2 to 3 min. Atlas cement was used showing a residue on a 200 sieve of 17 per cent. At the beginning of the work the mix used was of a relative consistency of about 1.25 corresponding to a slum of 6 in. This relative consistency was later reduced to about 1.10 corresponding to a slump of 2 in. to 3 in., which reduction in the water ratio was an important factor in cutting down the amount of laitance being developed.

Before the seal was poured the bottom was tested for mud with a small pump. This was given up, however, due to the awkwardness of handling the suction end in the bell and between the piles, in favor of sounding the bottom with a piece of 1 in. x 2 in. undressed plank to which the mud adhered, if the plank was slowly withdrawn from the water. This method was found to be a very effective one and some improvement was brought about by its use. Removing the mud by pumping failed to take out also the larger and heavier bodies, but the pouring of the heavy mass of concrete seemed to scour the bottom clean of any remaining silt and to get under and push up into the laitance mass both silt and loose but not

over heavy bodies such as old sunken timbers, etc. In the laitance was to be found crabs, eels, pieces of old timber and in one instance a 4 in. pipe cap that had been lost in the caisson the day before the seal was poured.

The $1\frac{1}{2}$-cu. yd. bottom drop bucket used for the depositing of the seal concrete was equipped with bottom doors, opening freely downward when tripped. The top of the bucket was open and was invariably full of concrete when lowered. A marker was put upon the line to indicate when the bucket was down, but this was an unnecessary measure as the slackening of the lines plainly showed when the bucket had touched the piles or

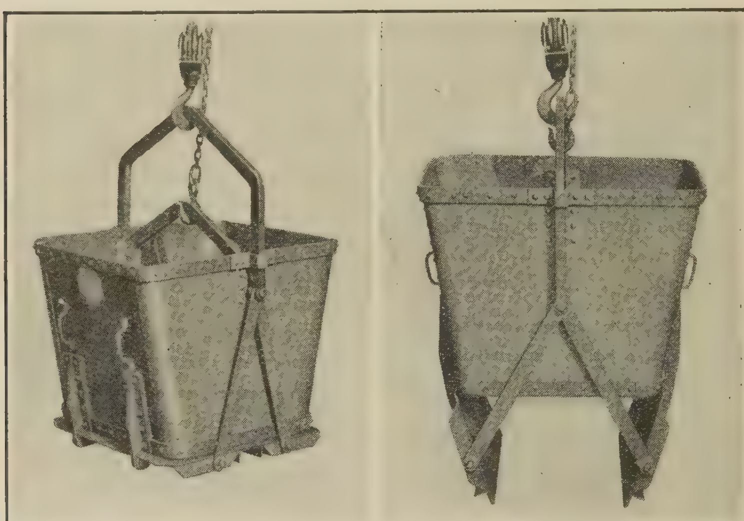


FIG. 2.—AUTOMATIC DROP-BOTTOM BUCKET USED ON COAL PIER CONCRETE WORK—CAPACITY $1\frac{1}{2}$ CU. YDS.

concrete. High piles that had been driven to refusal were invariably cut off before concreting. As the work progresses closer supervision was given to the speed of lowering and retrieving the bucket and to its position when discharged. The improved manipulation of the bucket by the derrick operator effected a considerable decrease in laitance.

The installation of an automatic trip at the bottom of the bucket, so arranged that the drops cannot be released until the bucket has touched pile tops or concrete, is a surer and cheaper method than continual supervision of the derrick. Careful supervision is, however, necessary to control the speed of the handling of the bucket if metal-top doors are not used.

Without doubt the somewhat too hurried lowering and raising of the bucket and occasional premature tripping of the bottom drops in the early

stages of this construction was the most important factor in the development of laitance during that stage of the work.

Causes of Laitance and Analyses.—An unavoidable element contributing to the formation of laitance and incident to all under-water pouring of concrete for foundations on piles was the 5-ft. drop of the concrete

FIG. 3.—FOLLOWING DOWN PIER FOUNDATION PILES.

through the water at the beginning of the pouring of the seal, caused by the foundation piles (See Fig. 1). However, it would have been impossible to sacrifice the stability of the structure by cutting the piles off at bottom for this factor.

From time to time as the work progressed, chemical analyses were made of the laitance, samples being taken from the top and bottom of the deposit. The dried and hardened laitance was also subjected to chemical

analyses but in no case was any marked difference to be found between its chemical constituents and those of the cement used on the work. By reference to the following analyses of laitance, Atlas cement before using, and other portland cements, it will be seen that its chemical constituents vary no more from those of Atlas cement than those of one standard brand of cement varies from another.

	Laitance	Atlas Cement Used on the Work	Standard Portland Cements—Hand Book Reference
Moisture	7.55	....	....
Silica (SiO_2)	22.49	21.31	19.06
Iron (Fe_2O_3) and Alumina (Al_2O_3)	7.56	6.84	9.76
Calcium Oxide (CaO)	62.40	62.80	61.23
Magnesium (MgO)	Trace	2.64	2.83
Sulfuric Acid (SO_3)	Trace	1.34	1.34

The laitance was found to be of a chalky color, hardening slowly, specific gravity 0.69 or of sufficient buoyancy to float until water-soaked. Samples of laitance taken out of the caissons in May, 1923, are still so soft that they can be easily crushed between the fingers. It appears to be a mechanical mixture of the finest particles of cement and a little fine sand and silt from the bottom. When the laitance is put through a No. 200 sieve and remixed with water a whitish mortar is formed smelling strongly of lime.

Laitance Quantity Records.—Accurate records were made of the quantity of laitance in each foundation. The yardage of seal concrete placed was recorded by batch record, and records by computation of volume of the seal concrete were also made. It should be explained in passing that the batch record is hardened concrete yardage computed from the yardage per batch of the concrete poured in the dry for which accurate cross-section records were made. Assuming that the wash of the charge of concrete, as it is deposited under water, affects only the outside of the charge and is in no way a re-mixing with the addition of water, then no additional swell in the concrete can occur. The batch record yardage should then be dependable. The computed quantities were from soundings taken on the soft bottom before the seal was poured. These data show the computed volume of the seal to be 96 per cent of the actual or batch record volume. Some small amount of concrete was squeezed out of the cylinder beneath the bottom ring but the difference is largely in the soft bottom displaced by the seal. Some small part of the silt pushed up by the concrete without doubt became a part of the concrete but the larger part of the volume was doubtless pushed up into laitance mass.

FIG. 4.—ERECTION OF STEEL CAISSONS AND BEGINNING OF PIER FOUNDATIONS.

FIGS 5 AND 6.—ERECTING A STEEL CAISSON (LEFT) AND WORKING IT INTO POSITION (RIGHT).

From data suggested in the foregoing paragraph and from certain reasonable assumptions following, an approximate percentage of the cement in the laitance will be arrived at. These figures are based upon data covering the entire volume of the work and not upon one particular foundation. This proportion makes no claim of absolute accuracy, however.

The difference in the total computed volume of seal concrete and the batch record volume can be assumed as the volume of silt or mud that was

FIG. 7.—POURING CONCRETE IN THE DRY.

scoured out and pushed up by the pouring of the seal concrete. Probably one-third of this volume became a part of the concrete itself and two-thirds or 81.3 cu. yd. became a part of the laitance mass. Using 29.5 per cent as the percentage of solids in the laitance, it follows that of the total volume of laitance 147.4 cu. yd. were solids. Deducting from this the 81.3 cu. yd. of silt, the remaining 66.1 cu. yd. of cement over the total yardage of laitance or 499.6 cu. yd. will give 13.2 per cent as the percentage of cement in the laitance. Attacking the problem from another

side—divide the unit weight per cubic foot of laitance or 43 lb. by the unit weight of cement that has been hydrated barely enough to give the same tensile strength as laitance, or approximately 98 lb., multiply by 29.5 per cent or the percentage of solids in the laitance, for a result of 13.0 per cent of cement in the laitance. It is therefore reasonable to assume 13 per cent as near the truth.

Even though 25 sacks of cement were added to the required amount for each seal to take the place of loss, at the worst the mix was reduced from 1: 2: 4 to 1: 2.8: 5.6. If the loss in tensile or compressive strength were merely what should be expected from the loss of richness of the mix the loss could be easily remedied by an additional allowance of cement. As the loss is of the fines of the cement it is a more serious matter.

Cement and Laitance Tests.—Cement is composed of a gradation of particles from an impalpable dust to a comparatively coarse clinker. Of this mixture the clinker or coarse particles are comparatively inert and only the finer particles or dust have quickly setting or binding characteristics, being, therefore, the most valuable portion of the cement. Taken separately neither dust nor clinker is effective but taken together a balanced mixture is formed. The dust in water forms laitance, while the clinker in water forms a weak mortar. If the two portions of the cement are remixed a greatly improved mixture results. To illustrate:

Into fresh water Atlas cement to one-tenth the volume of the water was dropped a little at a time and thoroughly stirred for some minutes and allowed to settle for three hours at which time the water had cleared up. On examination the deposit was found to be at the bottom a dark brown, almost black, coarse and gritty substance or clinker overlaid with slime or laitance. The volume of the resulting clinker and slime was found to be several times that of the cement used, as is always the case with cement agitated in an excess of water. Briquettes were then molded of neat laitance and of mortar using Ottawa sand in the proportions of 5 of laitance to 1 of sand, 4: 1, 3: 1, 2: 1, 1: 1, 1: 2, 1: 3, 1: 4 and 1: 5 and of the retempered mass or remixed clinker and fines. The results are shown in Fig. 8. Twenty-eight day tests showed the tensile strength of laitance to be 39 lb. per square inch; 1: 1 mortar 107 lb.; 1: 5 mortar 76 lb.; and of the retempered neat mortar 144 lb. per square inch. To further illustrate:

In 1909 Edward Godfrey of Pittsburgh found by experiment that if the fines or dust cement lodged upon the rafters of the cement mill were mixed in varying proportions with sand from 1: 10 mortar to neat, that between the two extremes a balanced mixture was reached. This balanced mixture was found to be 1: 2 mortar which showed a tensile strength of 855 lb. per square inch for 28-day tests compared to 165 lb. per square inch for neat mortar or laitance and 134 lb. per square inch for 1: 10 mortar. (See Fig. 8.) A further corroboration of this peculiarity of cement was found on examining buckets of laitance that had been removed from the foundations of Coal Pier No. 2 and allowed to dry out for

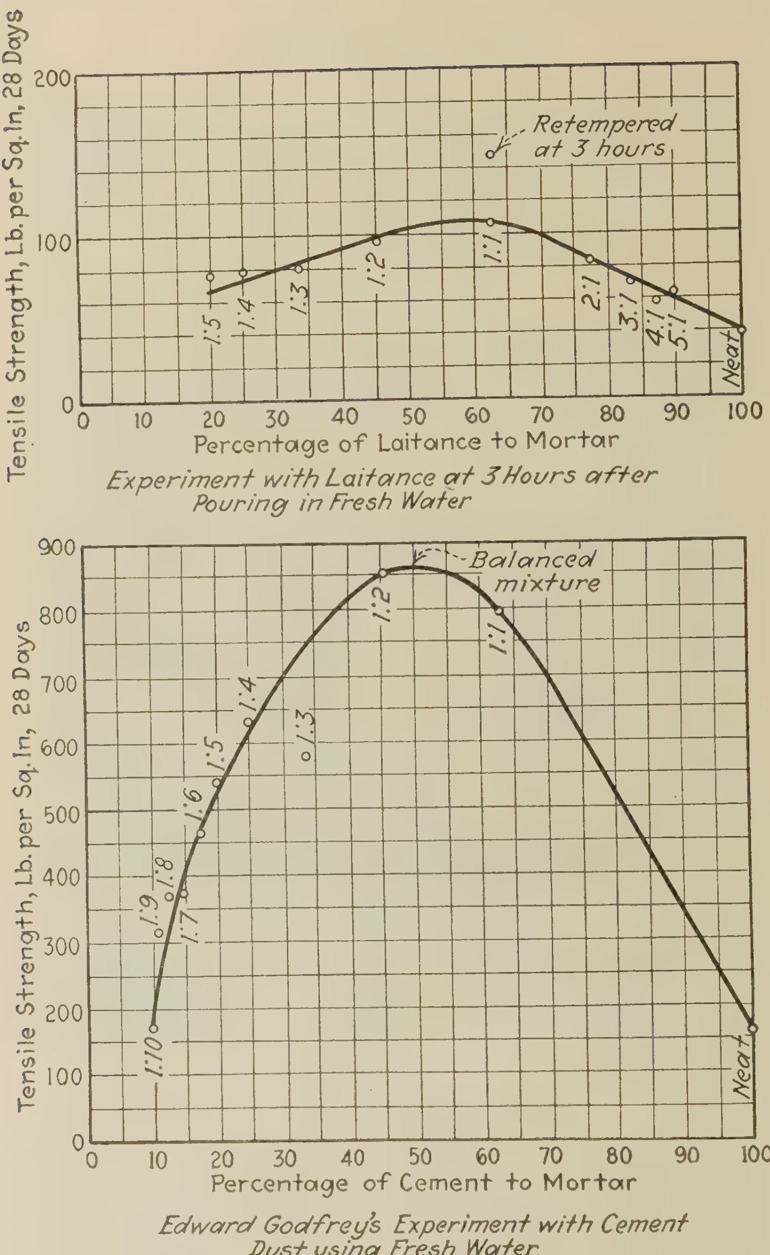


FIG. 8.—STRENGTH TESTS ON NEAT LAITANCE, SAND-LAITANCE AND SAND-CONCRETE BRIQUETTES.

several months. Small amounts of sand here and there were found in the mass of laitance. Whenever sand was encountered the laitance and sand zones had set into hard masses between which masses was to be found pure laitance that crumbled readily between the fingers. In other words, the sand and the fines had formed more or less balanced mixtures.

Observations and Suggestions.—Laitance, as has been pointed out, varies in chemical constituents from those of the cement used on the work no more than one standard brand of cement varies from another. It therefore seems unlikely that any chemical reaction or crystalization of importance takes place between sea water and the cement of the laitance during the pouring or during the four or five days that the laitance has remained immersed. In the mind of the writer the suspicion has lingered that the chemical constituents of sea water are responsible in some measure for excess formation of laitance in under water concrete. While this suspicion is strengthened it has been by no means confirmed.

Bates, Phillips and Wig, in Technologic Paper No. 12 of the Bureau of Standards, have shown that 1 : 2 : 4 concrete of Jersey sand and trap rock of a consistency called "quaking or mushy" when placed by a 5 in. tremie under fresh and sea water develops in fresh water at four weeks, three times the compressive strength of that poured in identically the same manner under sea water. Quite evidently something of importance takes place between sea water and the cement of under water concrete during the first weeks. On account of the conflicting views as to the behavior of cement in sea water it is not the intention of the writer to suggest the addition to the cement of siliceous materials or the use of special cements, but rather improvement through better protecting the plastic mass of concrete from contact with fresh or sea water by more careful methods of placing.

The formation of laitance withdraws from the concrete by wash the fines of the cement. It is extremely doubtful if the concrete placed by bottom drop bucket or tremie is so affected throughout its mass. It is highly probable that only the outside of the concrete charge is so affected and that the resulting concrete consists of masses of dense concrete between which will be found pockets or layers of concrete of much greater porosity. If the amount of permeability of the concrete is responsible for the rate of disintegration by the action of the salts of sea water and of the alkali waters of the West, it follows that the presence of layers or pockets of lean concrete or of laitance or construction seams invite failure. It is therefore of the greatest importance that the formation of laitance be held to a minimum. In the case of the Virginian Railway Pier No. 2 the steel caissons were left in place after completing the foundation. This is a very effective protection against the action of the sea water.

Whether or not any chemical reaction of importance occurs between the salt water of Hampton Roads and cement in the formation of laitance, the condition of the mix and the wash is in large measure responsible. It

is mainly a question of manipulation. Laitance can be held to minimum by the following:

(1) Use a clean aggregate.

(2) Make the bottom as free of mud as possible.

(3) Use fresh water in the mix, free from dirt, earth or sewage.

(4) Cut off all high piles that have driven to refusal. Hold the penetration of the foundation piles into the concrete to a minimum for stability.

(5) Use mix of a relative consistency of from 1: 10 to 1: 15 and use the slump test to keep the consistency uniform.

(6) Leave the bottom drop bucket open at the top but with bottom doors that open freely downward when tripped. Fill the bucket completely with concrete before being slowly lowered. When discharged the bucket should be returned slowly until clear of the concrete. The installation on the bottom of the bucket of an automatic trip so arranged that the bottom drops cannot be released until the trip has touched pile heads or concrete. If metal top doors are to be used they should preferably rest upon the charge of concrete when the bucket is full and open downward and follow the concrete as it is emptied. If the usual top doors resting upon the rim of the bucket are to be used it should be so arranged that they can be opened slowly upward before the charge is emptied.

(7) If the arrangement of the foundation piles and the shape and dimensions of the foundation caisson are such that the tremie can be used to advantage it is preferable to the bottom drop bucket, in the writer's opinion, provided the nose of the tremie is kept below the surface of the concrete already placed and the pipe is continually full of concrete. Every charge lost means additional laitance.

In passing it should be added that retempering of concrete has also been found of advantage in the elimination of laitance. This process consists in mixing concrete and storing for several hours during which time the initial set takes place. It is then returned to the mixer and remixed without the addition of water. Concrete retempered inside of two or three hours gains slightly in strength, hardens much more rapidly, will show no shrinkage after being placed in the dry, or swell when poured under water, and will not lose the fines of the cement through wash to any appreciable extent. English engineers extend the time of retempering to as much as five hours. Retempering of concrete on work of any magnitude in this country is of somewhat doubtful value as the time lost in storing and remixing is an important factor. The cost of furnishing either the necessary hopper cars and track for three hours' storage or the construction of

bins, chutes and an elevator or any other storage device would necessarily be reflected in the unit cost per cubic yard to such an extent as to make the plan on many jobs unacceptable.

The application of corrective measures to the pouring of the under water concrete for the Virginian Railway Pier No. 2 effected a marked improvement. Whereas on the extreme inshore end of the pier the formation of laitance reduced the mix from 1 : 2 : 4 to 1 : 2.8 : 5.6, on the outboard end of the resulting mix averaged 1 : 2.2 : 4.4. In short, if laitance develops either in under-water concrete or concrete poured in the dry the cause is too much water either in the mix or added to the concrete in placing.

Inasmuch as practical suggestions for the handling of a situation such as has been outlined are conspicuously lacking in technical literature, the writer offers the foregoing to the field man who is at times completely at a loss as to the causes and remedy for the apparently erratic development of laitance.

DISCUSSION.

Mr. Godfrey.

Edward Godfrey (*by letter*).—The discoveries of Mr. Miller on the formation of laitance on concrete deposited in water, and his recommendations regarding the handling of concrete to avoid or lessen the formation of laitance, are very interesting and of vital importance. The analyses that show that this substance is identical in composition with the cement and that laitance is not, as we are sometimes told, decomposed cement, are also of interest and importance.

Recognition of the facts pointed out by Mr. Miller is of the utmost importance. It is by recognition of facts such as these that progress in the handling of concrete is made. It is to be sincerely hoped that Mr. Miller's emphasis on these things will be heeded by engineers and particularly by authors of standard literature. He calls attention to the conspicuous lack, in technical literature, of the things he points out.

Mr. Miller shows by analysis that laitance is identical in composition with the cement; that the coarser particles of the best portland cement are totally inert; and that the laitance or finest dust of cement is the only cementing substance present.

That laitance or true cement is totally useless and hopelessly weak, unless it is mixed into the interstices of some inert substance, such as inert cement particles (too coarse to be true cement) or sand, are further facts which are explained in an article I wrote in *Engineering Record*, Oct. 16, 1909, p. 435, and re-demonstrated by Mr. Miller.

To the colloidal nature of cement Mr. Miller does not refer, but the facts demonstrate clearly that the strength of cement is due to its colloidal or glue-like nature and not to crystallization. Another paper to be read at this convention makes this statement, "instead of the lime liberating and crystalizing, thereby forming a reticulated structure which bonds the small particles of aggregates together," etc. This is said of lime which is also a colloid, and it illustrates a commonly held notion that the strength of cement is due to a mesh of crystals. This is a false notion, and intelligent use of cement is impossible until such notions are dissipated.

Not long ago an article by an expert in the chemistry of cement on the subject of laitance dealt with this material as though it were merely something of no value—on a par with mud—and the only problem was to get rid of it. The fact that this is the very product that is made by the undoubted improvement of cement manufacture—fine grinding—and represents large capital investment that produces that fineness, did exhibit itself in that expert's paper. The seriousness of the formation of laitance, in any quantity, because of the loss of cement value, and means of preventing its formation, did not seem to concern the writer of the paper.

Mr. Miller finds that concrete deposited in water may lose 13.2 per cent of the cement in the laitance. I have stated that it is well to use about 10 per cent extra cement for concrete that is to be placed under water to allow for loss.

Mr. Miller mixed his concrete 2 to 3 minutes, which is long for mixing concrete; less than one minute is very common. I have recommended extra long mixing for concrete to be placed under water, as does also the article to which Mr. Miller refers.

Retempering concrete or holding it for any time after mixing are both universally condemned and warned against in all standard literature and specifications, with no suggestion of any beneficial results under any condition. Mr. Miller found that holding for hours and retempering were very beneficial where the concrete is to be deposited in water. I have repeatedly recommended these prohibited practices.

A method of depositing concrete under water through a tremie, kept full of concrete and operated so that the concrete flows out the lower end as easily as possible, is not mentioned by Mr. Miller.

In view of the vast amount of testing done to discover what is the absolute maximum compressive strength that can be wrung out of a minimum content of cement, by the most artistic laboratory manipulation, it is pertinent to quote, anent Mr. Miller's discoveries, from my article, "If these phases of the nature of portland cement were emphasized in works on the subject, users of cement would no doubt learn to treat it more rationally."

THADDEUS MERRIMAN.—It is not necessary to go to under water placing of concrete in order to discover and study laitance. Much of it is to be found on nearly every work done in the open. On some jobs much more laitance is formed than on others, even if the same brand of cement and the same aggregate are being used. There is some as yet unknown condition which tends to cause the formation of laitance. The fact that it rises to the surface is an index of its low specific gravity, but the fact that it is seen on the surface should not be construed as indicating that no more of it is scattered throughout the mass of the concrete. It appears as though many of the cases where low strength is observed may be attributed to the excessive formation of this substance. The chemical analysis reported by the author shows that laitance is very much like the cement but the analysis itself is questionable in that it does not report either carbonic acid or insoluble residue. We have made many analyses of laitance in the endeavor to find a clue to the reason for its formation. Our analyses all show that it is saturated with carbonic acid and contains much insoluble silica which comes from the sand which is mixed with the concrete. **Mr. Merriman**

Last summer we made a large number of columns each a foot square and about five feet high, in the endeavor to introduce conditions which

would tend to produce laitance but without success. We have noticed that laitance seems to be absent from specially mixed test specimens but is nearly always present in concrete as it comes from a mixer. We have also observed that the quantity of laitance present is proportional to the amount of water used; that is, the more water, the more laitance. This may be due in part to the fact that there is a period during the process when the SO_3 concentration is below that necessary to control the set and that during such period the finer particles of the cement set up as individuals and so lose all of their cementing value and become inert. Another factor that has a bearing is the alkali content of the cement, and in this term we include only the hydates and carbonates of sodium and potassium. Calcium sulphate is more readily soluble in the presence of certain alkali concentrations than it is in others and so the "set" may be either accelerated or retarded depending on the quantity of alkali present. The average portland cement if thoroughly treated with an excess of water will show that the solubles are, lime 36 per cent, SO_3 29 per cent, alkalis 16 per cent, CO_2 19 per cent. The solubility of calcium sulphate in a solution of alkali depends upon the alkali concentration and so, as the alkali content of the cement varies, the protective influence of the calcium sulphate varies with the result that varying amounts of laitance are formed. This subject is well worthy of much study. If the entire story of the effect of the alkalis were known we would be a long way on the road to better cement and better concrete. Up to the present these very active constituents of cement have been almost completely ignored.

The following results of chemical analyses of laitance from the Gilboa dam are of interest. These results are expressed as the average of six separate determinations on six different samples. An analysis of the cement itself is included for comparison.

	Laitance	Cement	Laitance computed to same basis as Cement
Silica SiO_2	10.62	20.27	20.50
Iron Oxide Fe_2O_3	2.18	3.18	4.21
Alumina Al_2O_3	4.68	7.94	9.03
Lime CaO	29.93	62.09	57.76
Magnesia MgO	1.82	2.40	3.51
Sulphuric Anhydride SO_3	1.34	1.72	2.59
Loss on ignition	25.80	2.05	2.05
Insoluble residue	23.63	.20	.20
Alkalies	n. d.	n. d.	n. d.
	100.00	99.85	99.85

The loss on ignition for the laitance represents the loss after the sample had been dried for four hours at 100 deg. C. Approximately 90 per cent of this loss was carbonic acid and most of the balance was water of hydration.

T. P. WATSON.—To me there is no mystery about laitance. The presence of laitance is the result of the use of excessive water and it is practically impossible to avoid its forming on concrete placed under water. The excessive water content floats the light impurities in the cement, sand and coarse aggregate to the surface of the concrete and forms this scum or substance known as laitance. Mr. Watson.

Our experience with designed mixtures in the past six months has been, that we have not had any laitance form on our concrete placed in forms not under water. Regarding the method of placing concrete as outlined in this paper, I cannot conceive of the necessity of dropping concrete through 5 ft. of water. Anything might happen to concrete dropped through 5 ft. of water into the silty material usually encountered surrounding pile foundations.

C. A. P. TURNER.—Laitance prevents satisfactory bond between the newer layer and the older concrete. With a decrease in the relative amount of water used the amount of laitance brought to the surface decreases. In bonding the new and the older work it is better to remove the laitance before the older material becomes hard. It may then be scraped off and the surface roughened with a rake and a good bond for the next layer secured. Mr. Turner.

Material similar to laitance is developed where the concrete is allowed to flow and separate. Thus, if the workmen fill a beam box before filling the columns and allow the material to dribble into the column box separation occurs and concrete of little strength results which resembles surface laitance. It is about half as heavy as sound concrete and less than a third as strong and will necessarily have to be dug out and replaced for dependable material in the column core.

In sealing cylinders 40 to 60 ft. under water a foot of top material partly mud and partly laitance is sometimes encountered on the surface and then beneath this the concrete is sound and hard. This layer of inferior material results from the fact that the concrete being heavier than the light mud and at the bottom of the cylinder displaces it, forces it to the top along with the laitance and silt which, notwithstanding washing of the bottom with the water jet and taking out everything which the clam will bring to the surface, has been left to be displaced when the seal is made. There is great difference between brands of cement with respect to the amount of laitance and the care which must be used in splicing beams in building work. Sections a yard long wedge shape sometimes drop from the underside of the beam of their own weight where splices have not been made as they should have been in a vertical plane.

DUFF A. ABRAMS.—In general the author has properly stated the case of laitance, but I believe he has misinterpreted the curves in Fig. 8, dealing with what he calls a balanced mixture. The results found are due to the fact that he is testing briquets and that the strength of a briquet depends not only on the strength of the material, but also on its stiffness Mr. Abrams.

and rigidity. The dropping off in strength is due to the non-uniform distribution of stress across the section and does not reflect the true properties of the material. The tests of Mr. Godfrey cited by the author may be explained in the same manner. I do not believe there is such a thing as a "balanced mixture." Compression tests of concrete from all cementing materials show an increase in strength as the quantity of cement is increased.

A Member

A MEMBER.—Does the rigidity of the test specimen apply to alumina cement now coming on the market?

Prof. Abrams

PROF. ABRAMS.—Yes; our tensile tests of high-alumina cements both neat and 1 : 3 mortar show a retrogression in strength at periods of 3 to 7 days; in portland cement retrogression generally begins at 28 days or 3 months. Invariably the 1-year tests of briquets are lower than at some earlier periods. With the same cement in concrete you do not get that result, but a continually rising curve with increased age. This subject was gone into thoroughly in some tests carried out both in this country and England in a critical study of the stresses in a briquet; in other words, you get a tearing action that starts at the edge and works inward, and do not get the actual uniform distribution of stress across the section.

Forms for Reinforced Concrete Building Construction.

COLUMN AND FLOOR FORMS—EXAMPLES OF FRAMING AND RELEASING.

By J. A. Turner.*

Preliminary to a discussion of details in reference to the above subject, it is necessary that a summary be made of the general items of costs in connection with formwork. In general, the total cost of formwork to any contractor on any concrete job will be made up of the following items:

1. Cost of the lumber used: this is dependent upon
   - A Economical design of forms,
   - B Maximum number of uses of forms,
   - C Minimum amount of replacement after each setup, and
   - D Salvage value at end of job.
2. Labor expended: this cost is dependent upon
   - A Cost of making into the shapes or units to be used,
   - B Cost of setting up in position for concreting,
   - C Cost of taking down or stripping after concrete has sufficiently hardened, and
   - D After stripping, the cost of handling and hoisting and placing for the next setup.

It is evident that the details to be followed in making up the form units is going to have a very decided bearing on all of the above items. Forms so constructed that they can be easily released and reset will effect savings in both the amount of lumber consumed and the labor expended. Considering items 1 and 2, we find that in designing the forms they must fulfill the following requirements:

- A Substantial and reasonably unyielding but as light in weight as possible.
- B Few units as possible but no unit excessively heavy or cumbersome.
- C All corners and intersections of units so framed that the units can be easily released, saving labor in stripping and carpenter work in rebuilding. Solid square intersections must be avoided.
- D Provide vertical wooden keys at intersections of beams with columns and with girders. Where beams frame into girders, keys required

*Turner Construction Co., Philadelphia, Pa.

on one end only. Keys to be of uniform size in order that same can be sawed out on power saw.

The above points are those requiring careful study on all jobs. Now to get down to concrete examples, let it be assumed that we have a standard flat job of say eight stories with typical floors. Generally, one full floor of forms will be constructed and used eight times unless the floor area is exceedingly large and sufficient time is available to use forms twice

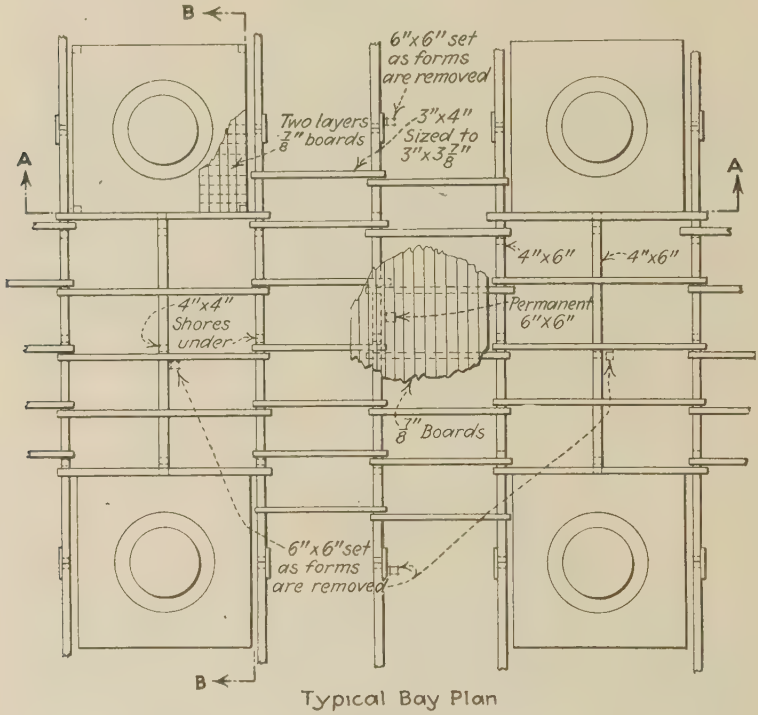


FIG. 1.—FORMS FOR TYPICAL BAY.

on each floor. However, sufficient vertical supports must be provided for several floors.

In the making of the one floor of forms it is necessary that the units be constructed in order that those that can be removed the earliest be framed to release ahead of the balance of the forms. This is essential to make the maximum amount of speed. The order of removal is as follows:

1. Column forms.
2. Forms supporting depressed sections around column heads.
3. Forms for exterior beams.
4. Forms for main slab.

The interior columns are usually of metal and the exterior of wood construction. The exterior columns are easily released if keys are provided around the beam sides intersecting same. The forms supporting depressed sections around column heads are usually supported on separate shores which can be safely removed earlier than the main slab shoring. The details of this portion of the forms are shown on Figs. 1, 2 and 3. These details can be varied somewhat, but the object must be to permit easy and rapid removal in order that this portion of the forms be erected

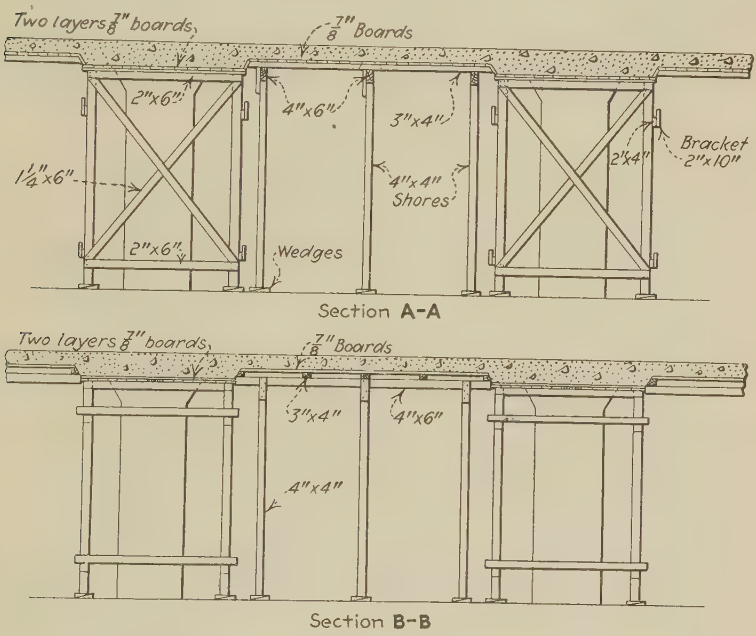


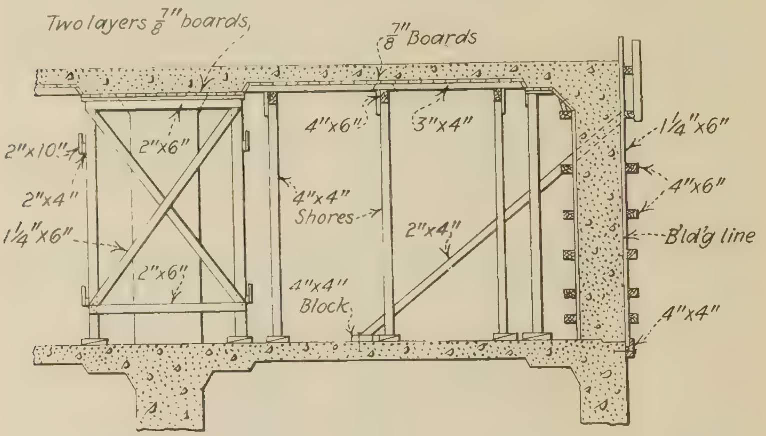
FIG. 2.—ELEVATION COLUMN AND SLAB FORMS.

for the next setup by the time the removal of the main slab forms is started. The exterior beam forms are easily released by removing the shores under one beam at a time and reposting immediately. To release this form quickly and economically it is necessary that vertical keys be provided at each end of the beam side where it intersects the column, and the bottom be cut slightly shorter than the neat distance from concrete to concrete of the columns and with a slight bevel, as detailed in Fig. 4. By this procedure the beam forms can be kept intact in one unit saving loss of material through breakage and mislaying, and also carpenter labor in erection. This unit can be hoisted and set in position by unskilled labor ready for the mechanics to secure in accurate location and replace the keys.

The supports for the main portion of the slab should be made up

similar to that shown in Figs. 1, 2 and 3. The 4 x 4 posts are handled singly, as are also the 4 x 6 capping plates or ledgers. These are hoisted and set up by carpenters ready to have the panels landed and placed in almost correct position by unskilled labor. The ⅞-in. decking is made into panels using the 3 x 4 spreader supports as battens. These 3 x 4's extend from one 4 x 6 to the next and when the panels are made up it is necessary to arrange for staggered spacing of 3 x 4's in adjacent panels in order that the 3 x 4's may secure full bearing on the 4 x 6 plates without fouling one another.

Panels should be made the length of the span from column to column, that is, to extend from the center line of one column to the center of the next. Panels between heads, of course, can only be the length from de-



Section Through Typical Ext. Bay

FIG. 3.—SECTION THROUGH EXTERIOR BAY.

pressed slab to depressed slab. A convenient width of panel and spacing of 4 x 6 plates is usually from 4 ft. 6 in. to 6 ft. 6 in. Beyond 6 ft. 6 in. the panels become too unwieldly and heavy for laborers to handle.

With the use of this system of formwork it is essential that a 6 x 6 permanent shore be placed in the center of the span before the concrete is poured to remain as the permanent shore when the 4 x 4 supports are removed. This 6 x 6 is placed directly against a small panel about 12 in. square cleated to the main panel in order that all forms except the small panel over the 6 x 6 may be removed without disturbing the 6 x 6. One bay of forms is removed at a time and an additional 6 x 6 is placed in the center of the band as soon as the panel is lowered to give permanent support for the slab at these points. This system permits the reuse of all of the floor of forms including the 4 x 4 shores except for the reposts under the exterior beams and the 6 x 6's. It also meets the requirements for economy

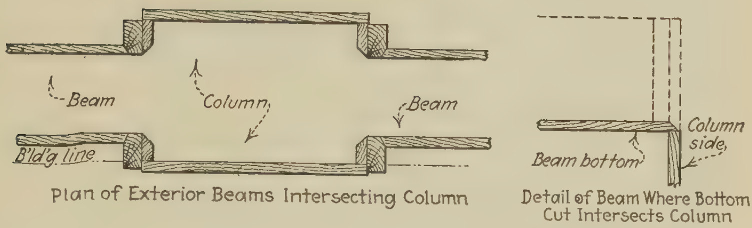


FIG. 4.—DETAILS OF BEAM FORMS.

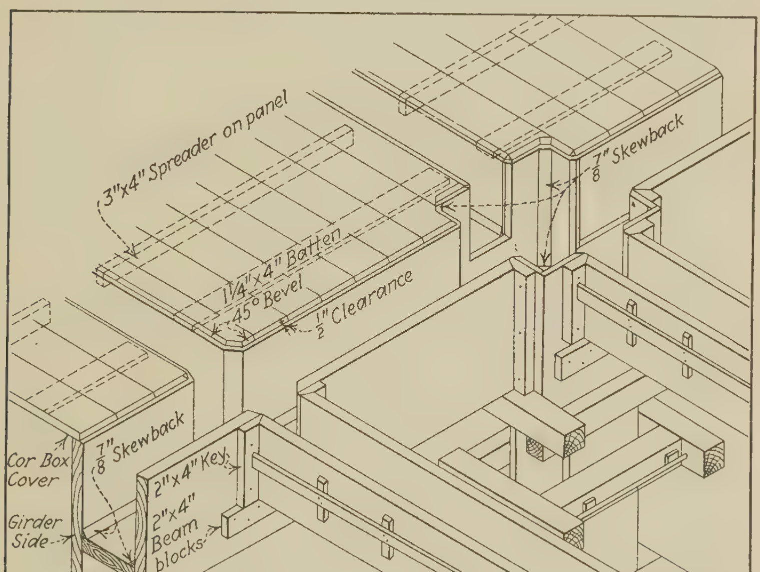


FIG. 5.—KEY CONSTRUCTION FOR VARIOUS INTERSECTIONS.

as outlined on p. 85. The unit cost per square foot for unskilled labor expended is slightly increased, but the expenditure required for skilled labor at high rates is materially reduced—unskilled labor not only performing all stripping and hoisting work but also doing a great part of the setting in place. A large saving is effected in lumber used, as the amount to be purchased for *replacement* after the forms are made complete for the first setup is almost negligible. As previously stated, however, the forms must be constructed with bevel cuts and keys in the beams in order that the forms may be released without excessive prying with bars, which is expensive both in labor used and lumber destroyed or damaged, requiring mechanics to patch.

Beam and Girder Construction.—Everything that has been said before applies more forcibly to beam and girder construction than to flat-slab construction. In beam and girder work there are more vertical points of contact in the forms and consequently more places to bind and more necessity for keys. It is also essential that the units be kept intact. If beam and girder sides are to be taken apart from the bottoms, labor must be expended to take them apart and carpenter work is created in the nailing together again. It is evident also that if the ⅞-in. decking is not made into panels much lumber will be lost or broken, requiring the purchase of additional lumber and the spending of additional labor to make the replacements.

It is an established fact that anywhere from 20 per cent to 40 per cent more lumber will be used on an eight-story beam and girder job if the columns, beams, girders and deck forms are not made into units and kept intact. It will be found that additional lumber will be needed to complete every floor of forms and on top of the purchase price of the lumber must be added the labor expenditure of receiving, hoisting and using of this lumber.

This wasteful expenditure can be eliminated by making the columns, beams, girders and deck panels unto complete units. But as stated above, these units can only be kept intact and readily released when stripping providing keys have been constructed.

Fig. 5 shows in detail the key construction for both the intersections of beams and girders and beams and girders with columns. The panels are made with the spreaders attached.

COLUMN FORMS.

By L. H. Usilton.*

In the design of column forms an effort is directed toward securing the maximum economy without injuring the appearance of the finished work. This requires as careful preliminary study as is given to the general procedure of the work. The plans and specifications must be carefully surveyed, materials purchased and arrangements made for storing and handling so as to avoid loss or damage.

General Requirements.—The requirements necessary to secure a good appearance are that lumber must be piled evenly so that it will not warp or twist; that in making, the different pieces be wedged tightly so that subsequent drying will not shrink, causing numerous fins to form; and that after making, an effort be made to keep the lumber damp to prevent such shrinking. Naturally, the different pieces must be of sufficient strength to take the concrete pressures without permitting any bulging or shifting. It is well to limit the width of sheathing to 5½ in. or 6 in., since wider pieces shrink a great deal more and leave heavier fins. Also, nails should not be driven home in the sheathing since subsequent wetting causes the wood to swell, countersinking the head and leaving protuberances on the face of the column corresponding to the nail head.

Costs.—Cost, in general, consists of labor, material and indirect charges whose importance depend on the influence the final form design may have on the remainder of the work. The direct labor and material costs are not difficult to ascertain but the matter of gauging the influence of the use of a certain type of column on the job may be very difficult. Those columns, cheaper in themselves, may cost more in the long run by delaying the progress of the work or by requiring extra work in making connections.

Direct labor costs are those involved in receiving and storing the necessary materials, handling to the job mill and performing the operations required there (boring, ripping, cutting), making into units suitable for easy handling, transporting to place, erecting, bracing and tightening, removing, repeating the handling and erecting cycle as often as necessary, then the final handling, breaking up and disposal.

Direct material costs arise from the purchase of lumber, nails, grease, bolts, washers, nuts, clamps, sheet metal and the various tools required to do the work.

Usual Design.—The usual design of exterior column form consists of vertical sheathing of tongue and groove material nailed fast to yokes

*General Superintendent, Barney-Ahlers Construction Corp., New York City.

of a size sufficient to take the pressure of the concrete. These yokes are usually 3 x 6 or 4 x 6 on the long dimension of the column and extend far enough past the form and yoke on the short side of the column to take bolts through holes bored in the long yoke, and yet admit a wedge to be driven between the bolt and short yoke close up to bring pressure on the short side. These forms are supported on brackets bolted to the pedestal or column immediately below, the bolts being stud bolts embedded in the concrete when poured previously. At the top provisions are made for connecting up with the floor and with the exterior lintel. All connections are made with small loose pieces called keys which, removed first, give the clearance required for easy removal.

Special attention must be given to the lining up of the top of the column where lintels come in on the outside and a bracket on the inside. Here bolts are not practicable and a vertical cantilever is used bolted through the top yoke lying below the lintel with the cantilever extending up to top of lintel to take the pressure of the concrete, and also running far enough down (one or two yokes' distance) to take care of any stresses arising. These vertical pieces are usually 4 x 6 and two per column.

It is usual to add small triangular or rounded pieces to all square corners in order to eliminate breaking off when removing forms. These vary from ⅞ in. to 2 in., depending on the size of the sections and to not appreciably lessen the concrete yardage. Also, small "recesses" are added to provide for sash, brick or tile to tie into. These are made with beveled edges so that they may be removed easily.

Column Design.—Interior columns are generally of round section with flaring capital and depressed slab section around the top. Forms are usually of metal and it is always necessary to take up with the metal form contractor the connections necessary for his columns. These vary considerably and provision must be made in the floor forms to take the head arranged for. These forms consist of metal sheets built to radius desired with rings or clamps of proper bend at sufficiently close spacing to preserve the lines of the columns. The flaring capital is made of various sheets and then fastened to the shaft and a ring placed at the junction. Some of these forms are supported on the lower floor; others are suspended from the floor form above. Provision must be made, however, especially to support the depressed head which may be carried down by the weight of concrete in pouring.

Variations in exterior column forms are made in many ways depending on the particular conditions existing on the job. Yokes may be left loose, battens being used to hold the sheathing together making lighter units to handle but more pieces to handle and perhaps lose. Instead of bolts, wooden clamps may be used with cleats holding at the corners and wedges added as the column is reduced. The metal clamps with a set screw gripping a rod may be used instead of bolt and nut. Adjustable metal clamps for any size of column may be substituted for yokes. These make a much lighter column and eliminate some labor in making up. Also,

on wide columns, in order to avoid very heavy yokes a vertical piece may be connected by bolts through the center of the column with another on the opposite side. Where columns are to be moved vertically the weight does not have much influence on labor cost. A winch will be used and a heavy column will come up as fast as a light one. Here there should be as few loose pieces as possible. Where, however, forms are to be lifted or handled by hand a study should be made as to just how they should be lightened, whether by loose yokes or by use of the clamps.

Interior columns may vary from the customary round metal to various shapes—octagonal, square or rectangular. Square or rectangular sections may be built similar to exterior columns except that the top connection to floor will probably be different. Octagonal columns may be made similar to square ones with triangular pieces fitted in the corners to bring in four additional sides. Round columns or columns approaching round shape may be made of wood. Full round columns are difficult to reduce. Where, however, the forms are made of an alternate tangent and curve with the whole keyed in place and clamped by a chain, it is easy to reduce by ripping part of the width from the tangents.

In designing interior column forms one point must not be lost sight of—when wood columns are used and are set up, a support true to line and grade is provided for adjacent floor forms. Where metal column forms are used these supports (usually wood horses) must be provided, thereby increasing the cost of floor forms. Therefore, where metal columns sometimes show cheaper in themselves the extra work which is charged to the floor may offset the advantage. This particularly applies to beam and girder jobs where round columns are used.

There are special cases of column design which may occasionally come up. Corner columns are usually L-shaped and are made similar to typical column with the corner blocked out. Columns against a wall must often be braced from the outside and stud bolts may be placed in the floor below to hold a ledger for such bracing. Also, expansion bolts may be placed in the wall. The wall in itself may need bracing against concrete pressure. Columns built monolithic with the wall usually bolted through in one direction, the other being braced against the adjacent wall.

Bracing is usually formed of level bracing in both directions to the adjacent column and diagonal bracing at least at the end of every row in each direction. Diagonal bracing may be to the base of the nearest column or to blocks secured by stud bolts in the floor.

Planning the Job.—In order to get an economical and straight job it is first necessary to take the plans and decide what set of forms is to be made up, then to figure each form move in detail, and what alterations are to be made. Occasionally concrete design may be altered to simplify this work. Forms are then planned to minimize alterations and handling. As an example, if columns are to be reduced in size it is well to make the side with strips which can be knocked off, thus eliminating hand ripping. Extra bolt holes should be bored in yokes by a boring attachment on

the saw so that they are not done by hand. Yokes should be laid out to minimize work when changes in the length of columns are made. On small jobs where there is no saw, recesses will sometimes cover a small opening in the short side of the column and save ripping.

Having gone into this phase thoroughly it is necessary to order materials to fit. Price will sometimes determine the decision, but in general clear, light woods are selected. Materials are received at the nearest point to railroad or road. Those to be milled should be moved towards the mill (which movement should always be toward first setup), then made, greased and piled neatly or moved into place. In building, all connections should be made with provision for bevels or angles to give clearances allowing forms to be removed and not wrecked, particularly if they are to be used again. After the construction plan is followed through it is necessary to wreck and clean up whatever is left. Cost of this cleaning up and the salvage value of material should be figured in the cost of forms.

In addition to the costs of material and labor there may be others to be taken into account. It would not be economy to save on column forms and lose a greater amount on another part of the job. For example, when metal form erection is let by sub-contract in addition to the cost of placing horses for support, the work of these men may be so slow as to slow up the whole job. Also, at the finish of a floor (forms) work may be a little slow for the carpenter gang and a number of columns may be set up before floor forms are ready to be removed. These are considerations which can be rated only as result of experience on other jobs.

Hard and fast rules cannot be laid down as it is only after a close study of all the requirements of a particular job that forms can be planned. Speed, type of finish required, character and supply of labor, price of various materials, prospect of other work, size of the job—all these must be taken into account. Sometimes, in haste, forms are designed to be put up with no thought given to taking them down, a costly procedure. There are comparatively few important differences in the construction of column forms which have much influence on the cost, the principal one being the use or disuse of any of the various clamps on the market. Contractors using clamps point out that when the cost of wood yokes—labor, cutting, handling, boring and making, of handling extra weight and of final wastage—are taken into account clamps prove cheaper. Others claim that initial cost, loss of parts, lack of adaptability, and the fact that more pieces are to be handled prevent their use. There can be no question but that certain types will work out splendidly on certain jobs. Whether wood yokes or clamps are desirable is a question which the superintendent and carpenter foreman must work out.

With many different cost systems in use it is out of the question to decide that any device or method is economical without first making a close study or perhaps a trial on a small scale.

Adjustable Shores Against 4 x 4 Wood Shores.

By E. C. Harding.*

A comparison between adjustable shores and 4 x 4 wood shores is a very complex subject. Each construction project presents different problems in shoring the forms. The type of construction, story heights, sloping floors, etc., will not admit of a general comparison. Shoring operations must be analyzed and studied. The unit operations will not vary materially on different jobs and different types of construction and a combination of these units will present totals that can be used for comparison. On account of these variables it is misleading at times to say that it costs a certain amount to shore formwork. The following data are presented for the various unit operations.

Initial Costs.

Adjustable Shores.—General speaking there are two methods employed to take care of the initial cost to each job. The shores are either rented to the job on a per diem basis or each job is charged a predetermined percentage of the initial cost. The initial cost is made up of the following: purchase price, depreciation, repairs, and a carrying charge. The purchase price depends upon the type of shore and will vary between $4.50 and $5. Depreciation is generally figured at 20 per cent per year. One manufacturer of adjustable shores has made an analysis of the cost of repairs and finds that over a period of six years and a total of 17,000 shores among seventeen users, 7¢ per shore per year is the average; this amount appears low and it is recommended that twice this amount be used. Six per cent is usually figured for carrying charge.

4 x 4 Shores.—When 4 x 4 shores are used the material is generally charged to the job and the usable material left is salvaged at a certain percentage of the first cost. The other items entering the initial cost are wedges and splicing material, labor necessary to make up the shore the first time, and the labor necessary to splice out or cut off the 4 x 4. The cost of lumber varies in different parts of the country. Wedges cost about 4¢ per shore setting. Splicing material from 10¢ to 12¢. To square both ends and cut to length costs 7¢. To splice a 4 x 4 costs from 35 to 40¢; to cut off 4¢. The above figures are based on $1.20 per hour skilled labor. The average life of a 4 x 4 shore is seven shore settings.

Erection Labor.

The erection labor per shore cannot be stated unless the type of construction, floor heights, sloping floors, etc., are carefully analyzed. A

*General Superintendent, Ferro-Concrete Construction Co., Cincinnati, Ohio.

comparison between 4 x 4's and adjustable shores on a large number of different jobs shows that adjustable shores can be erected at a saving of 8 to 10 per cent per shore erected.

BRACING.

When using adjustable shores, fully extended, proper bracing is very essential and additional braces are necessary over the amount required in using 4 x 4's. It is recommended that "X bracing" be used every 50 or 60 ft. both ways.

TYPES OF CONSTRUCTION WHERE ADJUSTABLE SHORES ARE NOT SUITED.

Adjustable shores can be substituted for 4 x 4 shores in shoring of forms in concrete construction. An analysis may show that 4 x 4's are as cheap if not cheaper than adjustable shores.

REQUIREMENTS OF AN ADJUSTABLE SHORE.

In conclusion it is deemed advisable to enumerate the fundamental requirements of an adjustable shore.

Long Range of Adjustability.—This is self evident and is necessary on account of the limits to be covered. The range should include at least 95 per cent of the shoring heights.

Safety in Use.—No contractor can afford at any time to endanger his work or employees. He must be very careful to see that the operation of the shore does not involve a personal equation. It is necessary that the device be so constructed that there is absolutely no danger of setting it so that it will not carry the load. The lock must preclude any doubt of the shore slipping when used in any and every possible manner for which it is intended.

Simplicity of Operation.—Adjustable shores are erected by any one from common laborers to skilled mechanics, a great many of whom are not mechanically inclined and do not readily understand the operation of a device of this character. Any mechanism which is not easily and quickly understood will be a serious handicap in its operation, and will materially increase the cost of its use. The same crew of experienced men is not always available and in many instances the contractor must call upon men who have never erected an adjustable shore. The device ought to be a complete unit, as the correlation of several units involves increased cost.

Sufficient Strength.—The device must be capable of supporting the superimposed load that it is designed to carry.

Weight.—The unit should be a one-man load, or seventy pounds.

Adjustability in Place.—This is an important feature as it is very often necessary to move the shore either up or down. The jacking device must be simple in construction and efficient in operation.

Construction of the Device.—The investment in adjustable shores is rather heavy. The unit must be so designed and constructed to withstand the wear and tear to effect the expected life and number of usings.

DISCUSSION.

B. P. Floyd (*by letter*).—The lack of standards is regrettable. The reason is perhaps obvious when one considers how few concrete buildings are alike. Also, lumber markets stock different sizes in different parts of the country. Comparison of costs mean little, due to lack of information in accounting systems. **Mr. Floyd.**

I am disposed to agree with Mr. Usilton of the Barney-Ahlers Co. that no hard and fast rules can be laid down for the proper forms for concrete buildings. Our experience has been that the difference between success and failure lies in the forms. A few general statements of what our years in the business have taught us may be in order.

Forethought, experience, and repetition in my personal opinion count most.

A very careful study should be made of every requirement of each individual job. Advance planning, even to minute details before any work is executed in the field, is extremely profitable.

To build formwork is not difficult. To get the maximum efficiency in formwork is another story. Experience has helped wonderfully in reducing our form costs. Our records are continually being bettered. Almost in direct proportion to the number of buildings erected do our superintendents rank on the basis of low units.

Everyone knows that the second time anything is done the performance is easier. I do not know where this applies better than on concrete forms. Have the forms always designed by the same man. Let the planning be one man's responsibility for all the work. A chosen crew of men in the field repeating the same operations year after year will bring almost unbelievable results.

Briefly, the Aberthaw Co. feels that no one contractor holds the secret key to success in building concrete forms. The time spent in study and applying the knowledge gained by close personal contact with many operations gradually develops the individual so that he soon becomes a master of the art.

G. E. Chamberlin (*by letter*).—On the building we have just completed for the Victor Talking Machine Co., which engineers have stated is as fine a building as we have constructed so far as straight lines are concerned, we cut all the 4 x 4 shores to an exact length and erected them without any adjacent section or wedges. Care was taken to get a level floor and in order to remove the shore easily it was set on a 1 x 4 strip. We secured by this method very reasonable costs, as well as uniform slabs and level floors above. **Mr. Chamberlin.**

In reference to dimension of yokes for column forms, our experience has been that if the speed of the job is normal and the columns of such a size that steel clamps may be used, considerable economy is effected in using the patent clamps instead of wood.

We note the suggestion is made in your report that framing of forms should be done at the mill, with which we thoroughly agree, except that we believe in keeping them in units small enough so that they will not be clumsy and awkward to handle, which is likely to involve considerable breakage.

The aim on every concrete job is cheap forms—low, ultimate unit cost in fabrication, handling, erection and stripping. We say ultimate because very often a seeming waste of labor in the fabrication of some part may be offset by the added number of uses obtained, or on the other hand, an apparent saving in erection costs may be thrown away in wasted labor or ruined material in stripping.

Saving must be effected in both material and labor, in the former by (1) economical size and spacing of various parts, and (2) design of the members for the maximum number of uses; and in the latter by (1) first cost of fabricating the units, (2) standardizing the procedure of erection and stripping, and (3) adoption of such devices as will allow use of the cheaper grades of labor as far as possible.

The above points must all be considered in relation to the job at hand; and any system adopted should have sufficient flexibility to cover all the needs of that particular job. For example, will there be a great preponderance of slab forms over wall forms, or vice versa?

What is the total form area of each type and what must be the speed of the job; in other words how many setups will be required?

May the finished job be comparatively rough, as in the case of a brick veneer, or must it be neat and true to line?

Here we will describe the system used on a large job where the bulk of the work consisted of forms for walls ranging in thickness from one to seven feet, and including foundations, tunnels, and finished exterior walls. The concrete pours would run from 5 to 20 ft. in height. The accompanying sketch shows a typical set-up.

It was considered undesirable to wire the forms, especially the exterior walls, and ⅝-in. plain round tierods with universal form clamps were used throughout. Wood spreaders were eliminated and removal of the tierods facilitated by the use of concrete spreaders. These were made of 1 : 2 mortar 2½ in. x 2½ in. square, in lengths of 6, 8, 9, 10 and 12 in., with a 1-in. hole through the center, and were reinforced with four pieces of No. 9 wire. As the tierod was put through the forms, spreaders to make the proper length were slipped on, and excellent alignment could be obtained as the spreader and tie acted at the same point.

Walers were made of two pieces of 2 x 6's, dressed four sides, separated 1 in. by wood blocks to allow passage of the tierods, nailed from both sides and clinched. Only the soundest and straightest of the 2 x 6-in.

stock was used. They were made in the mill in lengths from 12 to 16 ft. and spaced 3 ft. c. to c., proved very satisfactory in point of strength and service.

For studs, 4 x 4's dressed four sides were used 22 in. over center, in lengths of 8 ft. and up. These proved more economical than 2 x 4's as they allowed wider spacing on the walers with the maximum value of the face lumber, stood up well under rough handling, and if warped in one direction could be used equally well on the other face. No nailed splices were made, the stud lapping down behind the next waler.

For surface lumber both T. & G. and square edged was used. For panels, full sizes (15/16 in. x 5¾ in.) T. & G. in 14-ft. and 16-ft. lengths used. They were made with three 1 x 6 battens strongly nailed, 2 in. shorter than nominal length of boards and a uniform width of 3 ft. Outside edges were square. These sizes were not too heavy and were readily handled in erecting and stripping. For filling out 1 x 6 in. and 1 x 8 in. square edge, dressed four sides, was used. This material was bought in the same thickness and length as the T. & G. and was all run through the mill to the same cutoff length as the panels to avoid hand cutting in the field.

Hardwood washers 2 x 4 in. 9 in. long were used with the universal form clamps to give uniform bearing on both members of the walers. Nailed splices in the walers were avoided by setting 2 x 6's above and below the main waler, long enough to bear behind one washer each side of the splice.

The material throughout was standardized. A minimum number of parts was used and each part the least possible different sizes. The labor of erection was likewise systematized. Common labor supplied the material to the point of erection. To aid them, panels, walers and filler pieces after piling were marked with a color designating the length.

Actual erection also followed the same lines. Where a wall started from a footing a 2 x 6-in. kicker was spiked to the mat and the face form was always started with a 1 x 8 square edge; then followed the panels. Walers were spaced at the top of the 1 x 8 and at the joint between the panels. This added to the life of the panels, as after several uses, only the edge boards had to be replaced, and aided stripping as no rods were through the panels.

For staging, a portable bracket, as shown in the sketch, was developed and used with great success. They were securely attached to the walers as soon as the tierods were in place, and proved much cheaper than the ordinary form of knee-brace staging, as they will stand an indefinite number of uses and can be set up, removed and reset with common labor.

Mr. Harding.

Mr. Harding.—I want to present a few slides for your discussion. They are somewhat different in detail than the ones you have just seen that were prepared by Mr. Turner. This slide (Fig. 1) shows two different

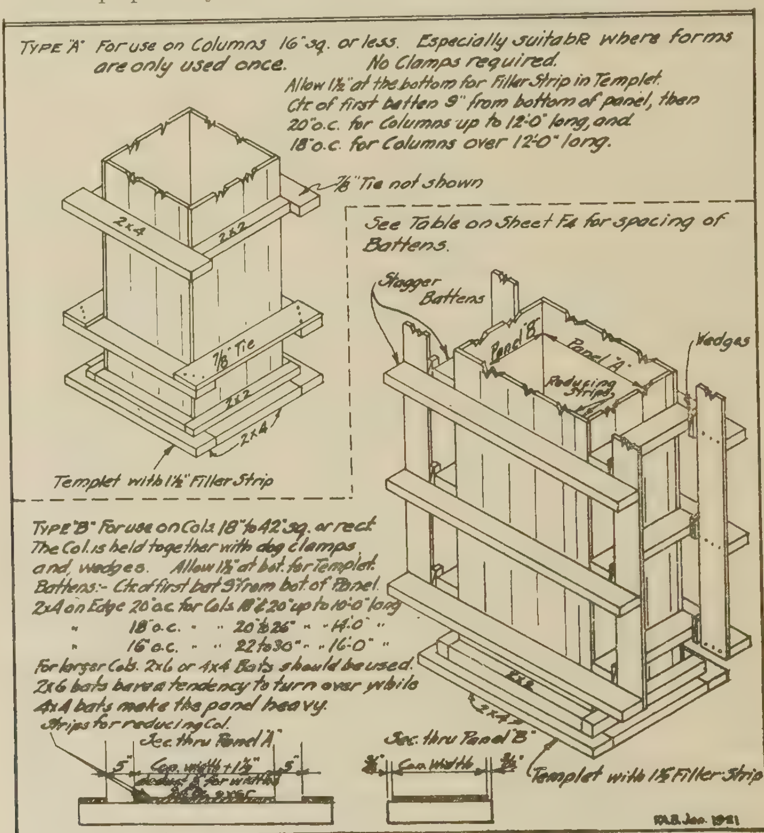


FIG. 1.—COLUMN FORM DETAILS.

types of column forms. This one on the left is to be used on small columns or columns that are not to be used any number of times. It is held together by ⅞-in. ties and 2 x 4 battens. The templet is made of 2 x 4's; an inch or an inch and a half is provided for a filler strip. The type on the right is used for larger-sized columns. You will notice on the filler panel, that the 2 x 4 battens extend ⅞ or ¾ in. beyond the sheathing. The making cost of this panel is slightly higher than if the 2 x 4 were

cut off flush with the sheathing, but stripping is easier and a truer column is obtained. Fig. 2 shows a larger column. It is made up with 2 x 2 battens in one direction on the wide panel and 2 x 4's flat on the other. The 2 x 4 battens are offset from the 2 x 2 battens. The filler panel has the 2 x 4's extending over ¾ in. on each side. It is held together by a ⅝-in.

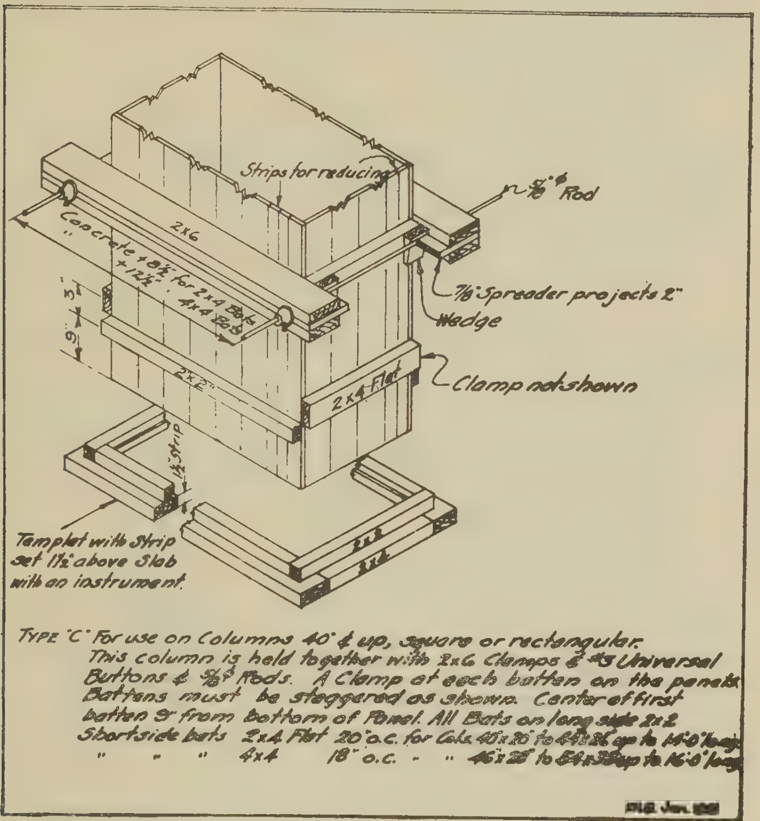


FIG. 2.—COLUMN FORMS OF LARGER SIZE.

rod with universal buttons. On particularly wide columns, the rods can be put through the middle of the column. The 2 x 6 clamps are nailed together with a ⅞-in. spreader that extends out some 2 in. to provide an additional back-up for the rod. We found where the ⅞-in. spreader was not used and where the wedge was ¾-in. from the edge of the 2 x 6, we got a little bending in the ⅝-in. rod. The 2 x 6 clamps can be used for practically any sized columns, although they are made up for a large column and reduced to a small, by inserting another piece of ⅞ in between.

Fig. 3 shows wall construction. The top view shows typical floor panels, which are made up four boards wide, usually of 16-ft. sheeting. Wires are used, or, where wires are not permissible, the universal button rod is used with double 2 x 6 arrangement. The pilaster is shown in the lower portion framed into the wall construction and over at the left is

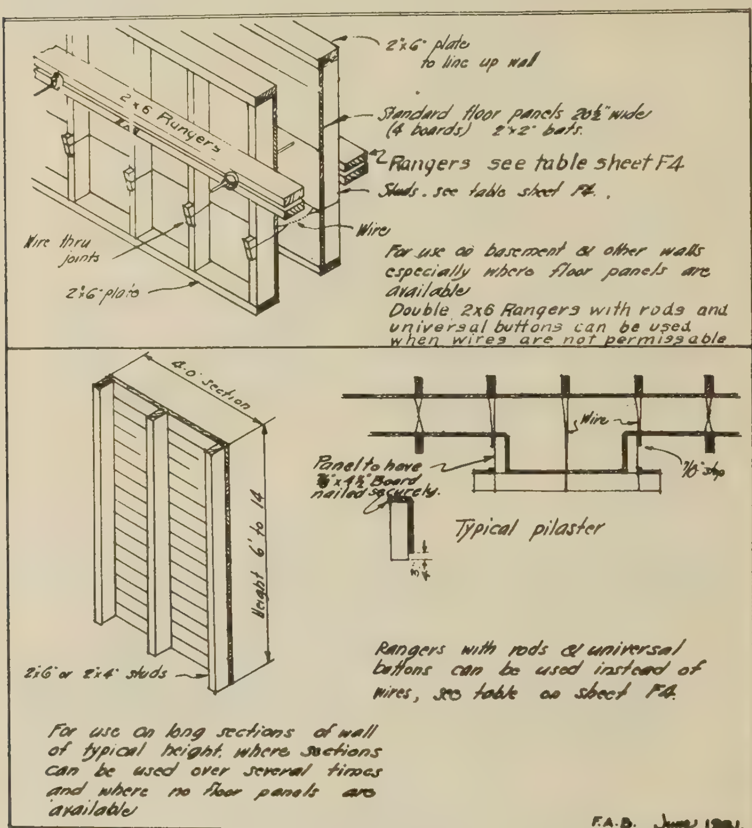


FIG. 3.—FORMS FOR PILASTERS AND WALLS.

shown a typical wall section, where forms are used a number of times or where floor panels are not available.

Fig. 4 shows our typical beam and girder construction. Notice that it varies considerably from the slides shown before. Columns are set up by the ordinary method. The girder bottoms are first put up, then the girder sides. The beams are put together as a unit on the floor and hoisted into position. The bottom of the girder is held together by a ⅞-in. tie or dog clamp, and usually the beam is held by a double headed nail from the

beam side into the bottom. On the beam side is a ⅞-in. joist bearer. The joists are single units. On top of them we start off with a bevel edge starting strip which is used also to line up the beams. The panels are usually three, four or five boards wide, depending upon the space required. Usually a filler strip is provided to take up any swelling that may occur

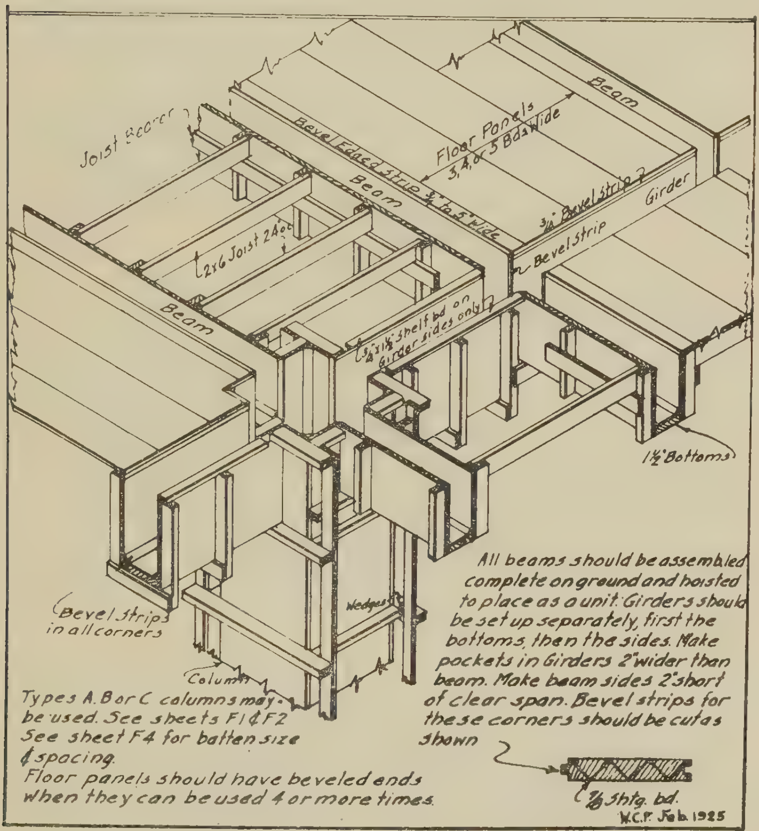


FIG. 4.—BEAM AND GIRDER FORM DETAILS.

in the panels. The girder side has a ⅞-in. shelf board about ⅞ in. x 2 in. that the floor panels rest upon. Any variation in length is taken up by that shelf board. The pocket in the girders is made 2 in. wider than the beam. The beam sides are 2 in. short of the clear span. This is taken up with beveled strips that are placed at the end to permit easy removal of the beam side. These are wrecked not as a unit, but as individual pieces.

Fig. 5 is used for the metal floor dome construction, either permanent or removable type. For the permanent type of dome we use either 2 x 8 or 2 x 6 rib bottoms. The lath, if it is used, is laid on top of these. The shelf strip is made usually the width of the concrete flange on the girder. Where we use removable tile the rib bottoms are cut exactly the width of the concrete rib and the method of supporting is shown in the upper corner. They are carried upon nails, a ten-penny nail driven in the side of the rib

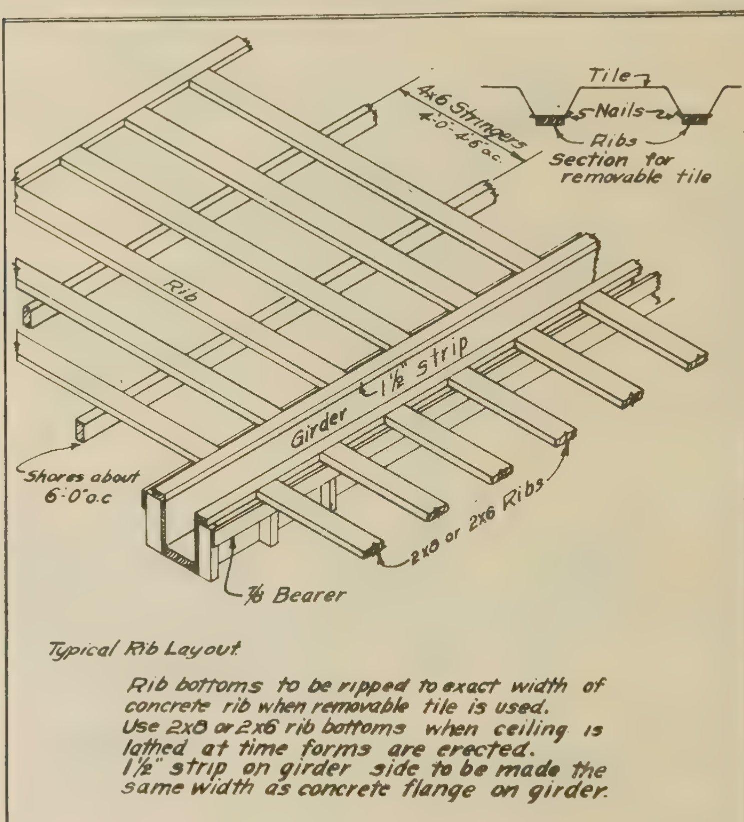


FIG. 5.—TILE AND RIB CONSTRUCTION.

bottom about three per length of tile, and where the lath is to be carried immediately under the joist, we drive nails into these bottoms. We usually drill a small hole, put the nail in there, let it stick up about half the length of the nail, and when the rib bottom is wrecked, the nail is in place. The lath is placed up against it, the nail is bent over and the lath held in this manner.

Fig. 6 shows our flat-slab construction. The columns are of the type determined. The templet for the exterior columns is shown in the upper left hand corner. The portion of the top of the column is a strip with either a stud bolt or a small bolt of some kind, and remains in position to carry the templet or outside column form for the succeeding story. The exterior columns are put up; the ledger and diagonal bracing installed and the shores and stringers erected. The spacing of shores, stringers and joist depends on the load to be carried. For the joists we use 2 x 6's put together in pairs approximately 24 in. on centers. They are spread by a 2 x 2 and held together by a twisted wire. These remain intact sometimes throughout the length of three or four jobs. Before the floor panels are started, the column head is placed. I will call your attention to the method of supporting it: We use hangers consisting of a 2 x 6 bolted to

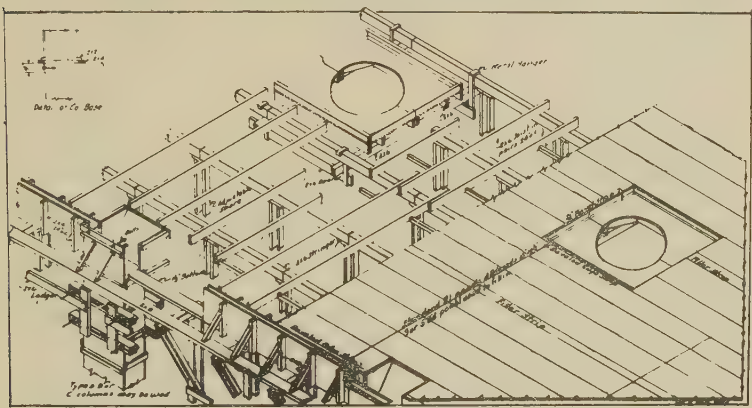


FIG. 6.—FLAT-SLAB DETAILS.

two irons. These irons are ½ x 4 in. bent as shown, with bolt holes, furnishing adjustment by steps of 1 in. A hanger is placed on two sides of the column. On these are placed the 4 x 6's, which carry the depressed head. These 4 x 6's are carried on wedges. The depressed head is built in the mill with 2 x 6 battens, bevel strip in corners, etc. It is made complete for the column and sawed into halves when first erected. The procedure in wrecking is to knock out the wedges, and this will permit the 4 x 6 to turn out and each half of the column head be removed. The floor panels are next started. We usually start with a beveled edge strip that can be of varying widths. We use ordinarily a four-board panel, about 20½ in. wide. Any difference in width is either made up in a five-board panel or a filler strip. This filler strip will vary according to the spans and can be ordinarily used for the re-shore placed directly against it, permitting removal of all the floor panels. The exterior beams are braced by the usual method of a T-head shore, shelf board, and lined up with 2 x 2's.

Mr. Roos. H. W. Roos.—Referring to the cost data furnished I would like to know whether adjustable shores or 4 x 4's were used.

Mr. Harding. Mr. Harding.—Adjustable shores were used on four out of five of the low cost jobs. The high price job—Job 19 was a 4 x 4 job; Job 11 was an adjustable shore job, and it was felt that the costs were materially lowered by using adjustable shores, in that decking was required at various elevations. Job 7 was also an adjustable shore job; the bottom of the slab varied as the slope of the floor. Adjustable shores showed a saving here.

Mr. Ahlers. John G. Ahlers.—I wanted to ask Mr. Harding if it was a conclusion or an inference from this report that the adjustable shores were not as satisfactory as the 4 x 4's, and also how he justifies the placing of the drophead on top of the floor forms so that they shall be the last thing stripped, after the floor panels are stripped, thereby delaying the erection of the steel column form?

Mr. Harding. Mr. Harding.—In answer to your first question—the point I tried to bring out in the subject of 4 x 4's vs. adjustable shores, was that you could not make a broad statement that one is better than the other, but that you have to take into consideration all the conditions existing on the job. I have made several analyses and can picture a job of, say, eight or ten stories, typical throughout, floor height the same, where 4 x 4's might show a lower cost than adjustable shores. On the other hand the basement is usually lower than the average, the first floor higher than the average, the second floor may be at a different height and the balance typical story heights. I think an analysis is absolutely necessary before you can determine whether 4 x 4's are cheaper than adjustable shores. What I tried to give was a few of the units that could be used for analysis. You know how many 4 x 4's you have to purchase; add to this the cost of squaring the end; the cost of material and labor for splicing and figure out the total cost.

Your second question about the dropheads being wrecked last—I thought I made it clear that it was wrecked first. This is made possible by the wedges under the 4 x 6 that is carried by the hangers. The wedges are knocked out and this 4 x 6 support from hanger to hanger can be turned over and withdrawn, and the column head wrecked in advance of the floor forms. It is erected before the floor panels, but wrecked first.

Mr. Grady. J. C. Grady.—In an adjustable shore, do you not have to lift the load to put the lock in place and if so, can it be re-shored?

Mr. Harding. Mr. Harding.—I am not familiar with all the types of adjustable shores on the market. We use two different types, one with an iron pipe and one with a 4 x 4. The 4 x 4 we do not attempt final adjustment with the locking device, we use wedges. With the other shore, it is not neces-

sary to throw any more than the load of the form construction to get it to catch.

Mr. Grady. MR. GRADY.—Can you use it for reshoring?

Mr. Harding. MR. HARDING.—I think enough pressure can be brought on the locking device to use it for a reshore; if not, it is a very simple matter to use wedges underneath it.

Mr. Ahlers. MR. AHLERS.—Has it been your experience that the tendency has been to increase the use of adjustable shores or to decrease their use?

Mr. Harding. MR. HARDING.—To increase it.

Mr. Foster. C. B. FOSTER.—I would like to ask if these costs, 9, 10, 13 and 4, include all labor, material, erecting and stripping? You said 9¢, 10¢ and 13¢.

Mr. Harding. MR. HARDING.—No, those just include labor on erection cost. What I tried to bring out there was a division of five erection items.

Mr. Miner. J. L. MINER.—I have just a suggestion to offer in connection with this cost of form construction. It is in reference to the possible use of alumina cement in certain forms of construction, relating to the earlier removal of forms and the more frequent use of forms. I think this is a suggestion worthy of consideration. In France, where alumina cement has been used for several years, it is reported that very marked economy has been effected on some jobs in the use of alumina cement in form construction. More recently in this country an example of what may be done on an apartment hotel in New York City, using the Eustis system of floor construction—the forms were removed entirely at the end of twenty-four hours. This was work done last December during fairly cold weather. It seems to me that it offers a suggestion or a possibility of saving certain types of work.

Mr. Immel. J. W. IMMEL.—Regarding the 4 x 4's and the adjustable shores, I would like to call attention to the fact that in figuring up comparisons, if you are doing work in several localities, out of a home warehouse, the items of loading freight and unloading are all but excessive on the adjustable shore. That fact must be given considerable weight in any comparison with the 4 x 4's.

Mr. Harding. MR. HARDING.—One of the jobs shown on the screen was what we would call an out of town job, where we shipped adjustable shores to the site and brought them back again and figured we had made money on them.

Mr. Priester. O. H. PRIESTER.—How often do you figure you can use 1-in. decking over on a job? Have you ever had any experience with 2-in. decking in flat-slab form, in particular?

Mr. Harding.

MR. HARDING.—We have used floor panels made of 1 x 6 D&M—I would be afraid to say how many times. I think a great deal depends on conditions on the job. Sometimes there are inserts in the floor. At the present time we have a job under construction that has inserts every few feet. Those floor panels will not last the life of the job, and they are only figured for three usings. Ordinarily I would say that these floor panels go throughout the average job and will go to another job for use in wall work. I do not know whether they can be reused, as in wall work they are usually badly cut up.

Mr. Priester.

MR. PRIESTER.—In other words, you do not know just how many times.

Mr. Harding.

MR. HARDING.—We have no records to show how long a floor panel will last.

Mr. Wright.

H. S. WRIGHT.—Providing you have a uniform floor surface, wouldn't you say that the wood shore would be preferable to the tension shore in stripping? I suppose you drop your shores about the same as every other man does, just pull them down with a hook from above. The impact of that metal shore on the finished surface might have some damaging effect.

Mr. Harding.

MR. HARDING.—The ordinary adjustable shore on the market today I do not think will withstand that abuse. That is one point I probably failed to bring out in the question of adjustable shores—that a little more care has to be exercised in the handling of them. The ordinary shore is made up of 2 x 4's in the upper member, and is usually a high grade of lumber and oftentimes will snap if thrown any considerable distance. That is one of the features that has to be looked for in the use of adjustable shores, in training the men to take a little care of them. In other words, they cannot be picked up and thrown down promiscuously like you would a 4 x 4, but I do not know that we have had any floors scarred from the wrecking. We have, however, had floors scarred during erection, in that you get on it usually before it is very hard. However, by the time you wreck your floor it is usually set up enough to withstand the ordinary dragging over it.

Mr. Wright.

MR. WRIGHT.—Don't you rather tend to raise your stripping cost because of the extra care you take? You probably strip with hooks or in some such manner as that.

Mr. Harding.

MR. HARDING.—It is probably apparent, but not real, and will not be brought out in any of the cost figures.

Mr. Christian.

H. A. CHRISTIAN.—Do you oil your forms?

Mr. Harding.

MR. HARDING.—We do, except where the concrete is to be plastered. The forms are oiled, usually, in the mill, and it depends on the condition of the panels as reused whether they are oiled again. Where concrete is plastered, we never use oil.

Concrete from the Viewpoint of Mr. Cement.

By Thaddeus Merriman.*

It is a great privilege, gentlemen, to appear by proxy at the bar of the Institute for the purpose of telling you, in the simplest possible language, a little something of myself, and of concrete, of which I am a close associate. Until quite recently I have been taken as a matter of course. "Cement," everybody has said, "is cement, and that's that." Yes, I am cement, and as such I am the heart and soul of concrete. Without me, concrete would be impossible. I am one of the most useful of our present-day construction materials, and I am literally at the very foundation of most engineering and architectural structures.

I am a well-made material, manufactured under conditions which are carefully controlled, and my quality is reasonably uniform. But I am not a definite single compound, and so, in some situations, I can produce results which I cannot duplicate in others. I have many troubles of my own. Most of them are the result of the over-zealous efforts of my many friends. To them I am a universal material, suitable for use anywhere, by anybody, and under any condition. In their eyes I typify the enduring qualities of the Rock of Ages. They speak of me as though I were the embodiment of every cardinal virtue. So it is that I have been misrepresented, and so it has come about that I am here to tell my own story and to ask that I be given only such credit as is my due.

I am the product of the rotary kiln and most people think of me as being a perfectly uniform and homogeneous material. But I am not so happily constituted. Some of me is good and some is indifferent as a cementing medium. Fortunately, there is enough of good in my makeup to mask and conceal a large part of my weakness. Some of my more conservative friends have recently been examining my anatomy with a view to determining how it may be possible to sort out and separate those parts of me which are best. I hope that they will succeed, because if all of the mediocrity in my makeup were eliminated, I could make good on the claims of the most zealous of my advocates. But enough of my constitution. Let me now tell what happens when I am mixed with water, sand and stone to make what is called concrete.

Just as soon as I am mixed with water I begin to hydrate. That is to say, I combine with the water and in so doing form what may be called a glue. This is how I am able to bind sand and stone together into a solid mass. But I cannot adhere to the surfaces of the sand and

* Chief Engineer, Board of Water Supply, City of New York.

of the stone unless I can come into direct contact with them. Sometimes also I cannot hold fast to these surfaces for the same reason that I cannot adhere to the smooth surface of a glass plate. Among the reasons why I cannot always get a firm grip are the following:

(a) The presence of adhering dust and dirt on the surfaces of the aggregate.

(b) The presence of loose dust and dirt which take up positions between me and the aggregate.

(c) The presence of inert material within my own mass which acts like loose dust and dirt.

(d) The presence of organic matter which adheres to the surfaces of the aggregate, or which, by surface tension, is drawn to these surfaces before I can reach them.

(e) The presence of a coating on the aggregate to which I can adhere but which itself has no bond with the aggregate, and, finally,

(f) The character of the surfaces of the aggregate. Some kinds of surfaces welcome, while others repel me.

It is usually said that by thorough mixing, these difficulties can be overcome because I am then to be seen as coating every particle of aggregate. I wish it were so, but, unfortunately, no amount of mixing will bring me closer to the surfaces which it is my duty to bind together. The eye is not a trustworthy judge. The only possible remedy for these conditions is to see that there is no dust and dirt present and that the aggregate is CLEAN. And by clean I mean that the surfaces of the aggregate should be not only clean to the eye, but so clean that I can make direct contact with them.

I have watched with much interest all of the vast effort which has been expended in the continuing attempts to secure stronger concrete by varying the sizes of the aggregate and by standardizing every operation except the most important one of all, which is that of giving me clean surfaces on which to work. Even standard Ottawa sand is far from having a standard surface, and so it is that sometimes I give much higher strengths than I do at others. Try me out with aggregates from the surfaces of which all dirt and acid soluble materials have been removed and you will be surprised at the results. Then clean the surfaces of the aggregates with an alkali solution and you will understand why it is that my shoulders are not strong enough always to bear the burden.

Even after I have succeeded in getting very close to the surfaces of the aggregate, I am confronted with another distressing condition, namely, that I cannot set or harden as long as the sulphuric anhydride concentration is above a certain point. This concentration grows steadily less as my setting time approaches, because the sulphur in solution is being precipitated out in such a form that some of it interposes itself between

me and the aggregate, and so the strength of my grip is lessened. Then, later on, as the concrete dries out the calcium hydrate and the alkalis crystallize and try to break my hold. My task, you will observe, is not an easy one.

Very, very often I find myself in the presence of so much water that I am, to all intents and purposes, simply drowned out. A reasonable amount of water is good, but enough is enough. In years gone by, when I was a hardy pioneer in this great land, I was not ground so finely, and, while my chemical constitution was less dependable, I could drink more water than I can now. As I have been improved, my ability to perform in the presence of water has been reduced.

I have already told you that I cannot set or harden in certain sulphuric anhydride concentrations. Now, if you will bear in mind the fact that I am a mechanical mixture of ground clinker and gypsum, you will have no trouble in visualizing what happens when I come into the presence of water. At first contact a considerable part of my finer particles instantly hydrate and set as individuals because the sulphur concentration requires time for its establishment. So it is that the finer I am ground the more of my cementing value is lost and the more sulphur is needed to control the speed of my reactions. It is true that while in the laboratory I show a somewhat greater strength the more finely I am pulverized, yet out on the job, which, after all, is my final resting-place, I could give a better account of myself if I were of a coarser grain. One of the worst things that happens to me is the agitation I get with the sand and the stone, not only in the mixers, but during the many motions I must endure until I realize the quiet of my final position. During all of these disturbances those parts of me which have hydrated or set as individual particles are rubbed off and displaced. They are no longer of cementing value and simply act to keep me away from contact with the surfaces of the aggregate. Laitance is the name by which those prematurely set and displaced particles are commonly called. That concrete which shows the least laitance is usually the most durable. All laitance does not float to the surface and the mass of many a concrete is honeycombed with it.

The simple fact of the matter is, that the more finely I am ground, the quicker will all of the cementing value within my mass be developed, and the greater will be my strength at twenty-eight days. My permanence and my ultimate durability, however, will be reduced because so large a part of my mass is converted into useless and inert laitance. Now, under certain alkali concentrations this laitance swells greatly and operates to increase my bulk and so to reduce my density. In consequence, the concrete of which I am a component absorbs large quantities of water during every rain, and the water so absorbed dissolves out the swollen laitance which is highly soluble. As time goes on and I am wet by more rain, I become more and more honeycombed, and, finally, I am myself dissolved away because I, too, am soluble.

The point I would like to bring most strongly to your attention is that

those concretes which show the highest early strengths are, not in general, those of greatest ultimate permanence and durability. No one ever yet succeeded in lifting himself very far by the use of his bootstraps alone. In my makeup I have just so much and no more of cementing value. By grinding me finely you can get more of this total cementing value developed at an early stage, but my later strength and my durability will both be lessened. I am no exception to the rule that the same cake can be eaten but once.

I could tell you much more of my inherent peculiarities, but my limited time will permit me to refer only to one other point, namely, that of the strength of the concrete of which I am an integral part. Without me, this same concrete, no matter how its ingredients might be proportioned, would have no strength at all, it simply would not be concrete. No concrete has as much strength as I have myself, because I cannot take hold of the aggregates as strongly as I can adhere to myself. Hence, as time goes on, and as my hydration becomes more and more complete, I grow stronger but little. So it is that all of your compressive tests seem to show the concrete as growing stronger day by day, while if tested in tension, that same concrete would show almost no increase after twenty-eight days.

Inasmuch as I am the basis of all concrete, and am responsible for its behavior, I hope that the compressive test will be abandoned and that a real test will be substituted for it. Never in all of my experience have I seen a concrete in real life fail under compression. Failure always manifests itself in tension cracks. The science of concrete can advance but little if the sole criterion of its quality is to be expressed as a compressive strength at twenty-eight days. There is no known relation between durability and compressive strength. No matter how strong a concrete may be in compression, its tensile resistance is far too small to meet successfully the stresses it must bear. And this is amply evidenced by its cracking everywhere. Better concrete will come when you learn how to develop all of my strength and my adhesive qualities. The compressive test will never show the way; nor will any investigation which is based only on mechanical and visual examination. If you wish to learn more of my secrets, it will be necessary to adopt that type of abstract reasoning and deduction which has given us the spectroscope, the election relay and the autogenous vaccine and has made possible far-reaching advances in nearly every branch of science.

Up to this time you have studied me only with your eyes and with your hands, but not until your hearts and minds are summoned to the task will you come to know me as I am.

Notes on Construction of Concrete Stadium.

By W. K. Hatt.*

This paper describes the concreting of the Ross-Ade Stadium at Purdue University which is named after David E. Ross, of the class of 1893, now trustee of Purdue University, and George Ade, of the class of 1887. The Osborn Engineering Co. of Cleveland, Ohio, designed the stadium and was represented by an inspector. An engineering committee of the university advised the authorities. The contractor was A. E. Kemmer, member of the American Concrete Institute, a graduate of Purdue University in the class of 1902.

The stadium is U-shaped with the half circle and a portion of the straight sides constructed on an excavated clay bank, covered with 6 in. of cinders. The last 100-ft. lengths of the open ends of each of the two wings are supported on columns and girders offering space underneath for the dressing rooms. The stadium fits a natural basin and is so located as to balance the cut and fill.

The stadium will eventually seat 23,200 persons. At present the straight sides are complete with seats for 13,400 persons. The half circle end is excavated and terraced with cinders to afford standing room.

The excavation was begun on June 2, 1924. The field was sodded, the concrete complete ready for the most important game of the season on Nov. 22, 1924. The approximate quantities were 50,000 cu. yd. of excavation and 3,000 cu. yd. of concrete.

There had been an unfortunate failure of a nearby stadium. Two other cases of damaged stadiums were in evidence. The situation was such that the university authorities were apprehensive. Those concerned in the construction desired that everyone connected with the job—contractor, foreman, workmen—would realize the necessity for carefully manufacturing the concrete. It appeared useful to exhibit to the foreman photographs of these failed structures, and to make field tests of the concrete from time to time and communicate the results. The compressive strength of the cylinders were exhibited as fast as they became available, contrasting the strength of wetter and drier mixes and showing the benefits of proper curing.

The materials were portland cement, washed torpedo sand (F.M. 3.2) and pebbles from ¼ in. to 1 in. (F.M. 6.8) in the proportion 1 : 2 : 3. The standard cement mortar tests (1 : 3) averaged 229 lb. per sq. in. at 7 days and 321 at 28 days in tension, and in compression, 2 x 4 cylinders,

*Professor of Civil Engineering, Purdue University.

2,067 lb. per sq. in. at 7 days and 3,416 at 28 days. The moisture in the sand as tested was 4.2 per cent and in the gravel 2.6 per cent.

The mixer was of ½ yd. capacity without time-lock or automatic water control. The operator had had eighteen years of experience in mixing concrete.

Cylinders were made at intervals from July 22, 1924, to Oct. 16, 1924, to the number of 114. Some were exposed on the job and some cured in damp sand.

Fig. 1 shows the running conditions throughout the period of construction. The operator of the mixer used his judgment in pouring the

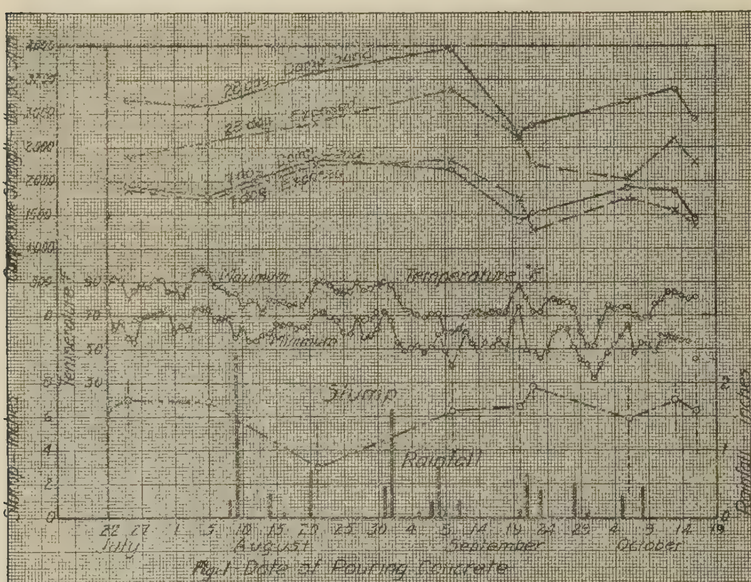


FIG. 1.—RUNNING CONDITIONS THROUGHOUT MIXING PERIOD.

water. When the concrete did not work well during placement, word would come back to him to change the water. The samples of concrete taken during any one day were formed into twelve cylinders. For each cylinder a slump record was obtained.

In Fig. 1 the slump value represents an average of twelve determinations; the daily range is indicated. The values of strength represent an average of three cylinders. It will be noted that as the wetness of the concrete was reduced the strength values increased, and that the curing in damp sand added very substantially to the strength of the 28-day cylinders. This showing of the benefits of curing brought about a favorable attitude on the part of the foreman, so that the deck was subjected

to running water as early as possible. By the careful curing and protection of the surface by canvas, subsequent effects of contraction in the deck are believed to be largely eliminated.

In Table I it will be seen that the average slump was reduced from 6½ in. on July 24 to 3 in. on Aug. 20. It is considered that the effects of this dry concrete are not only shown in increased strength but increased durability and diminished danger of cracking. Of course such drier concrete needs careful slicing in thin deck risers and treads. Some patching must be expected. My impression is that in portions of the work a slump of 3 in. meant an unreasonable dryness of concrete.

As the work proceeded the concrete became wetter due to the necessity of rushing the work, and possibly the conviction that the exhibited results of strength tests indicated an unnecessarily strong concrete. At

TABLE 1.

Date	Slump, in.	Strength			
		Exposed		Damp Sand	
		7 days	28 days	7 days	28 days
July 24, 1924	6½	1600	2400	1650	3200
Aug. 20, 1924	3	2200	2800	2250	3500
Sept. 22, 1924	8	1500	2250	1550	3000

any rate, the slump ran up to 8 in., daily average, on Sept. 22, with the decreased strength of the concrete shown in the table.

Fig. 2 shows the customary variability diagram of the strength of the concrete and a comparison with Jobs A and B reported by Messrs. Stanton and Walker in the *Proceedings* of the American Concrete Institute for the year 1924. These Jobs A and B were 1 : 2 : 4 concrete, 28-day cylinders. There was much closer supervision and better-controlled apparatus on these Jobs A and B. A comparison of the slump on this Purdue job with Jobs A and B cited is shown in Fig. 3.

Fig. 4 shows the relation between slump and strength. Each point is the record of one cylinder and its corresponding slump.

It will be noted from this figure that 1 : 2 : 3 concrete at 6½-in. slump average yielded 3,250 lb. per sq. in. in damp sand at 28 days. Abrams' tables for 1 : 1.8 : 2.9 concrete with from 6 to 7-in. slump, 0-4 fine aggregate and 4-1 coarse aggregate indicate a strength of 2,000 lb. per sq. in. Possibly the strength of the cement explains this difference.

Expansion and Contraction.—This construction offered an opportunity for measurements of expansion and contraction. One hundred and six gage points were set in parapet walls and different portions of the deck, both on the excavated bank and in the open section. Thermometer wells were inserted at various depths in the concrete. Observations of change of length and temperature have been made from time to time, and will be

continued with the hope that representative values of thermal coefficient of expansion for the several exposures and of the amount of contraction due to shrinkage and of expansion due to absorption of moisture will be obtained.

Parallel measurements are made of length changes under laboratory conditions. The complete record of the laboratory investigation is found

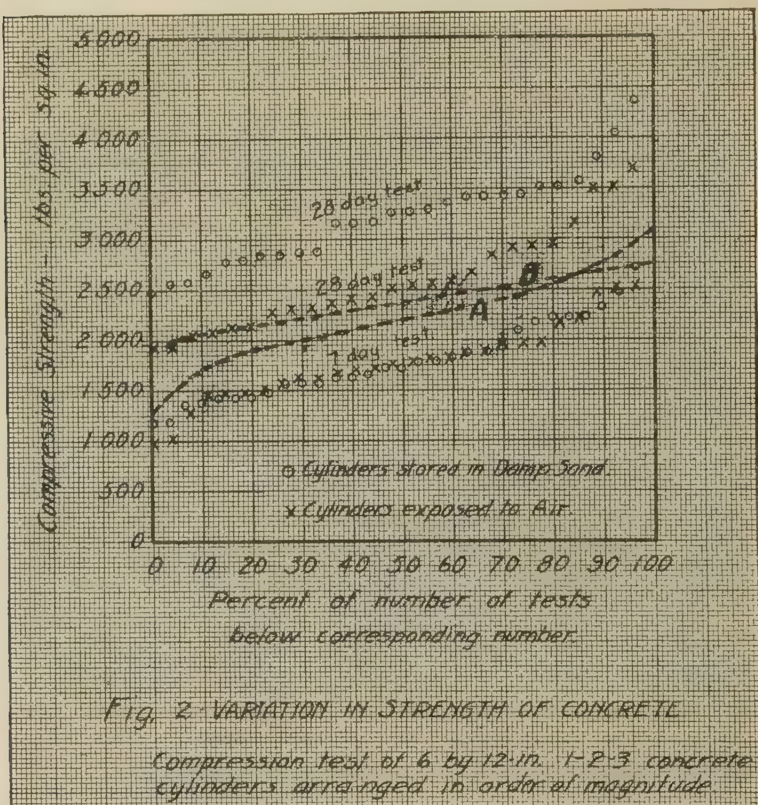


FIG. 2.—VARIATION IN STRENGTH OF CONCRETE.

in a paper by the writer entitled "The Effect of Moisture on Concrete," presented at the annual meeting of the American Society of Civil Engineers January, 1925.

Some of the measurements on the stadium may be recorded here.

The complete laboratory investigation just cited indicated that the thermal coefficient of expansion depends on the amount of moisture in the concrete, and also on the temperature of the concrete. An average value

for 1 : 2 : 3 concrete may be taken as 0.000005. (See Fig. 5.) Fig. 6 shows the behavior of beams of the stadium concrete exposed outdoors. Fig. 7 shows the contraction of several brands of cement compared to D, the cement used. Figs. 8 and 9 show length changes of concrete beams.

The shrinkage of concrete depends, among other things, upon the richness. A 1 : 2 : 3 mix may be expected to shrink eventually to the

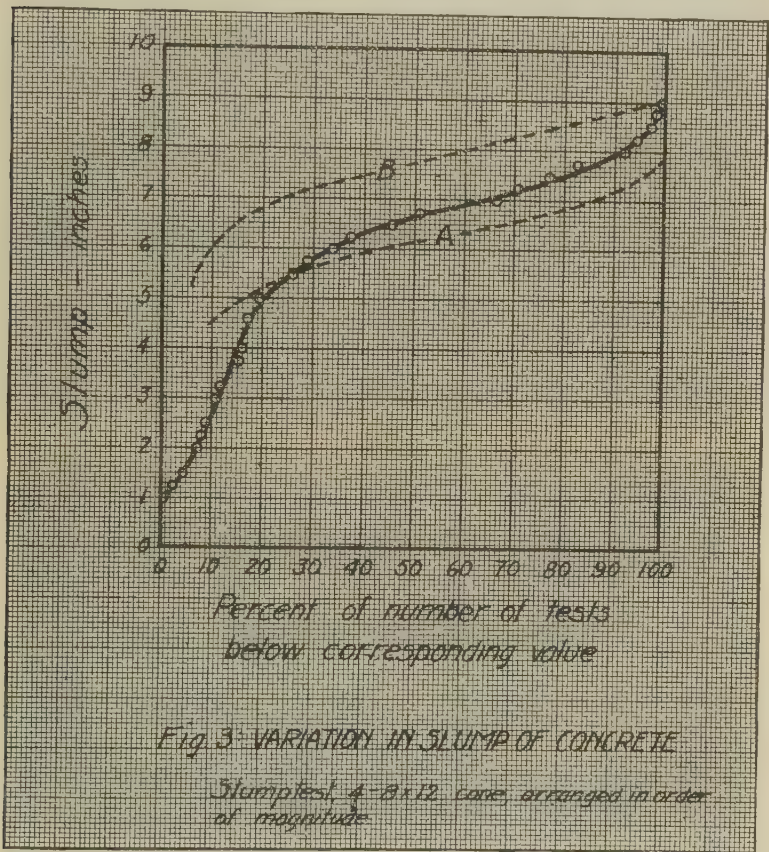


FIG. 3.—VARIATION IN SLUMP OF CONCRETE.

amount of 0.05 per cent as a result of measurements begun two days subsequent to the pouring of the concrete. During these first two days the greater part of the shrinkage may be expected unless the concrete is carefully cured. If the concrete is not protected from early drying, then this value of 0.05 per cent will be exceeded.

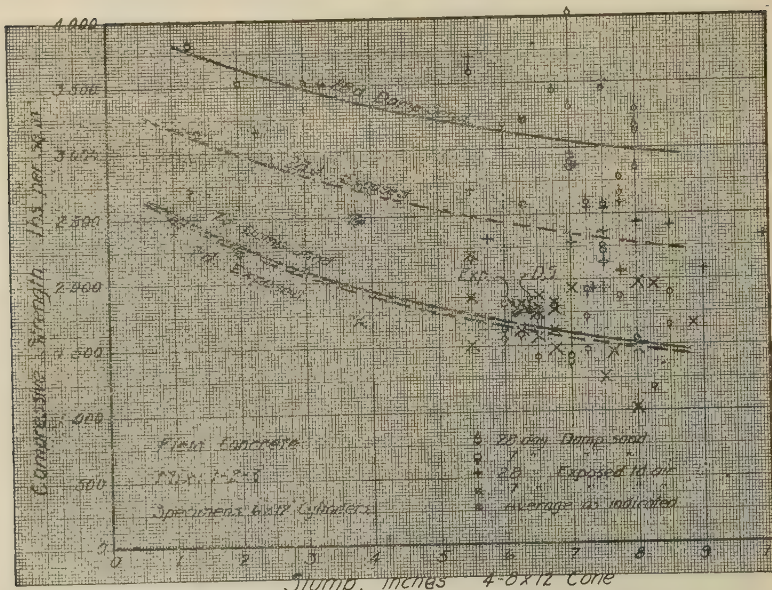


FIG. 4.—RELATION BETWEEN SLUMP AND STRENGTH.

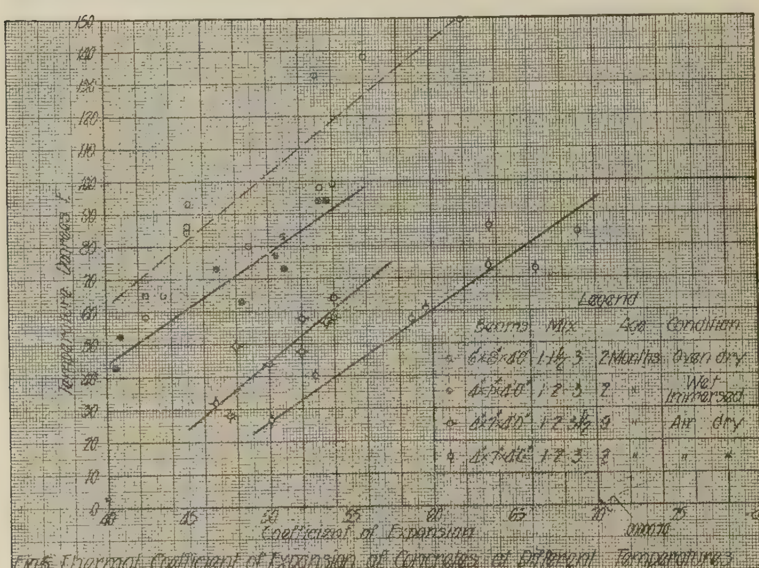


FIG. 5.—THERMAL COEFFICIENT OF EXPANSION AT DIFFERENT TEMPERATURES.

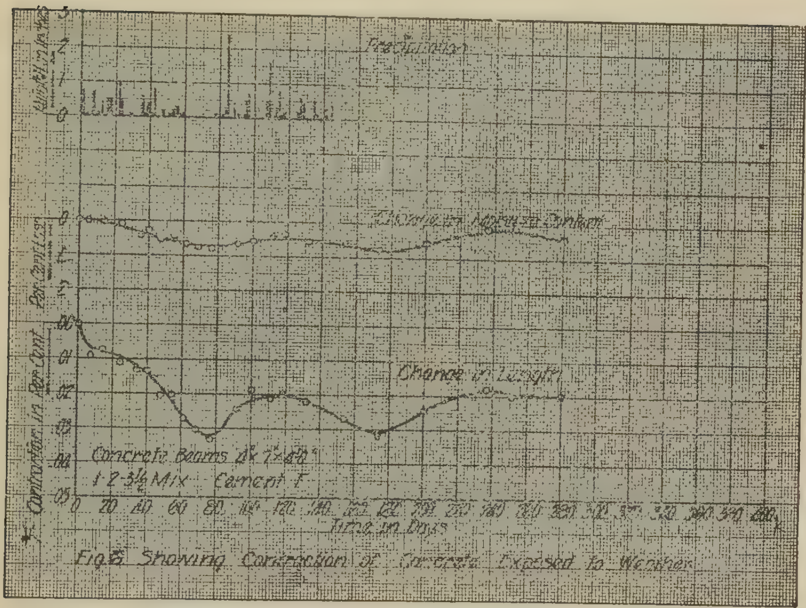


FIG. 6.—CONTRACTION OF CONCRETE EXPOSED TO WEATHER.

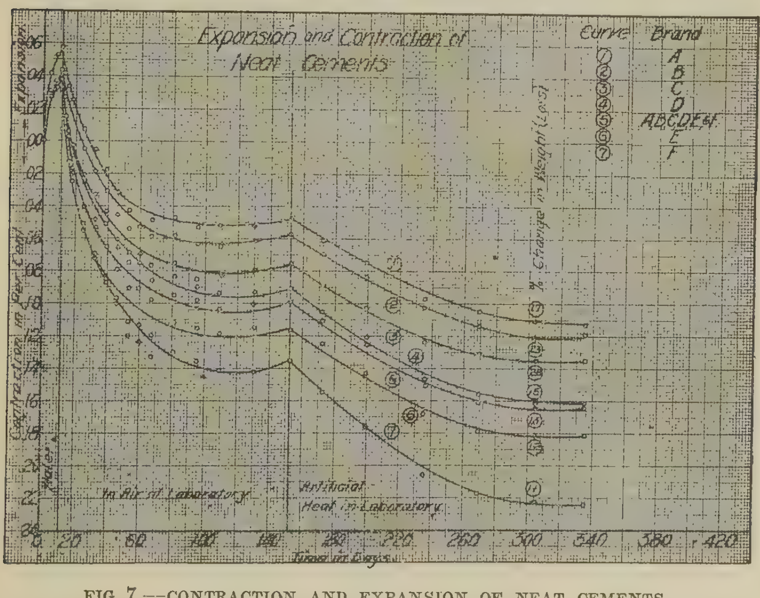
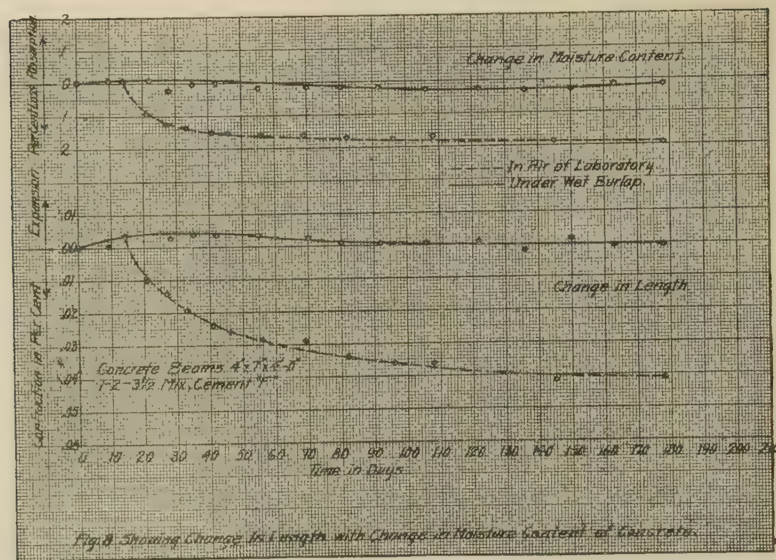
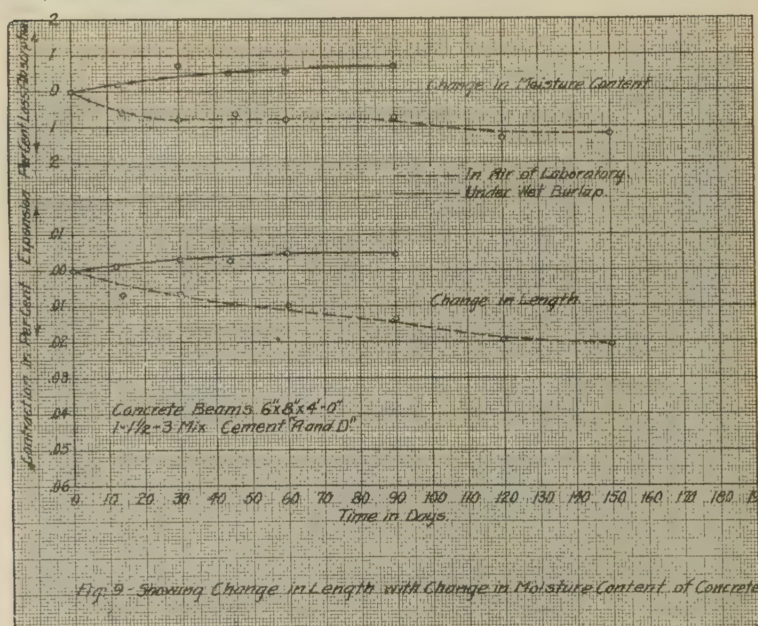


FIG 7.—CONTRACTION AND EXPANSION OF NEAT CEMENTS.





FIGS. 8 AND 9.—CHANGE IN BEAM LENGTHS WITH CHANGE IN MOISTURE CONTENT.

Absorption of water may increase the length of dry concrete in the amount of 0.01 per cent. Under ordinary conditions of exposure this increase in length may be taken as 0.005 per cent. Measurements of the behavior of the stadium to date show that under a drop of air temperature of 38 deg. F. the concrete in the open-air deck section of the stadium

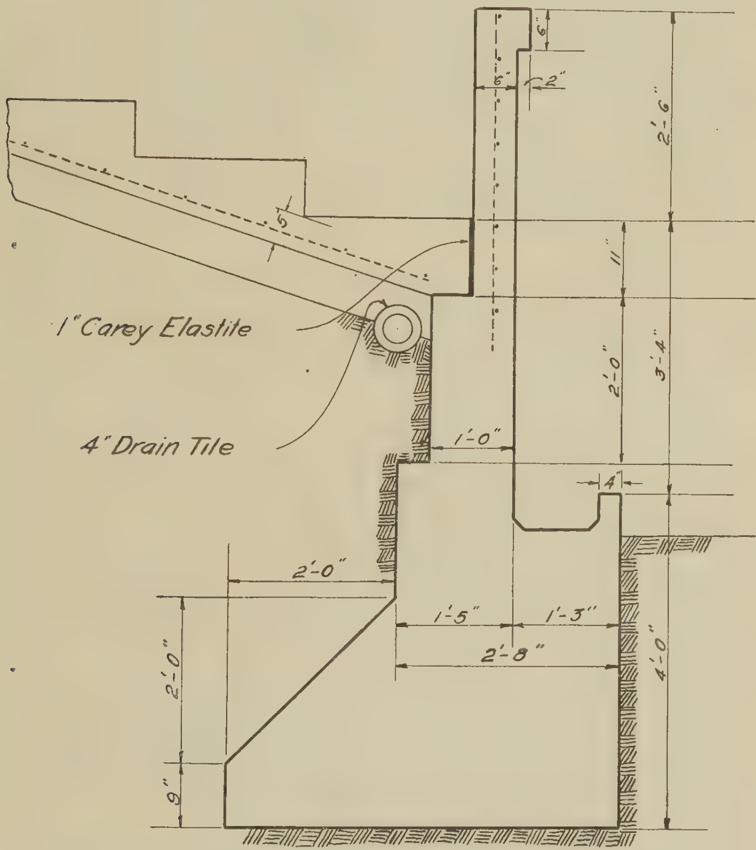


FIG. 10.—PARAPET WALL SUBJECT OF OBSERVATIONS.

dropped 37 deg., the concrete on the bank 24 deg. The thermal coefficient of the open deck was 0.0000059 and of the revetment 0.0000049. The difference is probably due to the varying dryness or wetness of the concrete. A rainfall followed by a drop of 10 deg. F. balanced each other, and no change of length was recorded in the deck corresponding to a thermal coefficient of 0.000005 and a swelling of 0.005 per cent.

Interesting observations were made on a parapet wall shown in Fig. 10.

This parapet wall was 6 in. thick and approximately 3½ ft. high and 6 in. thick reinforced at the center of its depth with 0.3 per cent of steel each way. This thin wall was integral with a heavy base. Naturally the thin wall with its shrinkage and exposure to sudden drop of temperature and held at the base was under considerable tensile stress. The steel reinforcing apparently was not enough to prevent a fine crack in the middle of this parapet wall, that is, half way between the aisles which were on 45 ft. centers. The thermal coefficient of contraction of the top of this wall was 0.00000725 per deg. F. following a drop of air temperature of 38 deg., which appeared at a drop of 28 deg. in the concrete of the wall. This same coefficient at the junction of the parapet wall with the base was 0.00000305. The parapet wall was poured mostly in hot summer weather, and the crack appeared after the cool nights set in. These cracks, however, closed up entirely during the winter weather, no doubt because of absorption of moisture.

The maximum drop in air temperature from early fall to January was 52 deg. in the concrete. There was a difference of as much as 10 deg. in temperature between the sunny side of the parapet wall and the shady side. The opening of this crack in the middle of the length of the parapet wall was 0.000471 in. per degree drop in temperature.

The expansion joints in this structure were made by placing a ¾-in. expansion joint material against the end of the finished section and pouring the new concrete up against this expansion board. The latter appears to be too thick and stiff to hold itself in line. These expansion joints are now all open. Where they run parallel with the axis of the seats, water penetrates through the deck to the clay bank and to the team rooms, so that these very narrow openings between joint material and the face of the concrete will require filling with some undetermined material. It is a question if it would not have been better in a job of this kind to form construction joints, permitted to be open a slight amount and to be afterwards filled with an elastic filler.

Conclusion.—As a result of the experience of an average job of this kind, much can be gained by demonstrating to the foreman the results of tests of the concrete. When foremen come to an actual belief in the effect of variations in their practices, they are much more likely to co-operate with the engineer in securing good concrete.

The variation in quality of concrete on this job is so much greater than variation in the constituent materials that evidently increased uniformity is to be sought in the actual mixing and placing of the concrete.

On a job of this kind in this vicinity we have very little doubt about the uniformity and quality of the material. Attention must be paid to the manufacture and placing of the concrete and subsequent curing. Slump tests made under the eye of the mixer, the results of compression tests communicated as fast as they accumulate to the foreman and an early protection of the concrete will certainly largely diminish any hazard there may be under local conditions in pouring a large outdoor structure like

a stadium. These structures require more care in selection of materials and in manufacture than concrete not exposed to atmospheric influences.

It is gratifying to notice the extension of technical control of materials and manufacture to structures erected by large organizations that include engineering talent. There is, however, a large task to be performed to translate the wealth of scientific information to the average job of concreting. Much of this information must be simplified, freed of forbidding calculations that seem intricate, and attention concentrated on a few simple tests for dangerous aggregates, and the essentials of concreting. These should also be performed with a purpose of educating the concreting gang.

DISCUSSION.

Prof. Slater. W. A. SLATER.—I understand from Prof. Hatt that the coarse aggregate used was No. 4 to 1 in.; is that right.

Prof. Hatt. PROF. HATT.—The fine aggregate was No. 4 sand.

Prof. Slater. PROF. SLATER.—What was the maximum size of the coarse aggregate?

Prof. Hatt. PROF. HATT.—One inch.

Prof. Slater. PROF. SLATER.—What was the size of the beam that it had to go into?

Prof. Hatt. PROF, HATT.—The treads, as I recollect, were 5 in. thick.

Prof. Slater. PROF. SLATER.—I asked the question because I know that in the Illinois stadium we also had some difficulty with some honey-combing on the sides of the beams. We had generally a width of four, and in some cases, five inches for the beams and had a coarse aggregate of limestone with a maximum size of about three quarters of an inch, (specified to be five-eighths, but there was a good bit of it that ran over), and I have thought that for that kind of work that is rather too coarse an aggregate.

Mr. Abrams. DUFF A. ABRAMS.—Prof. Hatt spoke of the non-uniformity of the concrete on this job. It seems to me that his tests point to the fact that it is extremely difficult to get representative samples of concrete for testing. I do not believe these tests necessarily mean that concrete is not uniform, but in taking a sample for test from a batch, you are likely to get something different from the average concrete in the batch. We find that many mixers do not discharge the concrete uniformly and that, of course, increases the difficulty of getting uniform samples.

Prof. Hatt. PROF. W. K. HATT.—I think what Prof. Abrams says is true, that the variation in masses of concrete cannot be as great as the variation in small samples indicates, but finally, our judgment must be on the technique of making these tests. The point I want to make is this, that our materials do not vary greatly, our cement tests run very uniform throughout the seasons and our washed sand and aggregate are uniform also. If you do not find variations in quality, I think the room for improvement must be in the manufacture of the concrete; there is where we should concentrate our attention.

Mr. Miner. J. L. MINER.—Just to add a word to what Professor Hatt has said of the effect of variables on the results obtained. During the measurement of the change in volume of "lumnite" cement concrete, that is the change in length of concrete specimens, it was noted that the drier mixtures developed a greater increase in length during the early period of hydration with a smaller shrinkage at subsequent periods. With more mixing water there was a less increase in length with a greater shrinkage. These observations confirm what Professor Hatt has stated regarding the effect of the amount of mixing water on the change in volume during hydration of portland cement concrete.

THADDEUS MERRIMAN.—Prof. Hatt referred to one very interesting and very important matter, which is that of keeping concrete wet during the early hours of its life. The first ten or twelve hours are usually the most critical. The practice now followed on our work is to begin the application of water just as soon as possible, after the concrete has been placed, and keep the concrete continuously wet for twenty-four hours thereafter. The essential result sought is that of stopping volume changes and giving the mass the opportunity of taking its final form and attaining such strength that it can hold itself together when the shrinkage stresses come upon it thereafter. In former days when cement was not ground as finely as it is now, concrete could be placed and allowed to remain over night and forget it and come back next morning and it would be all right. Since finer cement has come, however, unless the greatest care is exercised during the first night, irreparable damage will often be done. Mr. Merriman.

Crazing on Cement Products.*

By P. H. Bates.

The title is purposely chosen because the three common products using cement as a bond all show crazing. This difficulty is not restricted either to neat cement or mortar or concrete. So far as cement products are concerned we are all familiar with the manifestation of crazing. Some believe they know the cause of this and a still smaller number believe they know how to prevent it. (Parenthetically, they are eligible for membership on this committee provided they will demonstrate facts and not present theories.) Consequently this paper will not discuss crazing objectively but metaphorically only subjectively. Furthermore, it should be borne in mind that while I am occupying the time allotted to Committee T-1 this discussion is the product of my own thoughts, and the members of the committee should in no wise be adversely criticized for any of the statements contained herein. Any other criticisms will of course be highly acceptable and appreciated by them.

Crazing is not new. It undoubtedly developed in the first product made by the original consumer of Joseph Aspdin's personally made portland cement, assuming that he not only first patented it but also first made portland cement. But crazing is much older than this. Our hypothetical first parents, if they were as observant of other natural phenomena as they were of the fruition of apples, undoubtedly noted it in the first dried up mud puddle they came across. As we are all convinced that the mud puddle preceded man on this sphere, so we must acknowledge that crazing is older than humanity. When man decided to improve upon the habitation furnished him by nature and build himself mud houses he found this occurrence of nature following from the puddle to the mud houses. When he started making utensils or writing materials of dried or partially burned mud he again found crazing an inherent difficulty. But when he tried protecting surfaces of burned clay products with glazes, or surfaces of wood or steel, etc., with paints or enamels he found crazing so persistent that he practically assumed it was inevitable and accepted it without any question or further thought. This attitude maintained for centuries and even today certain makes of pottery are in demand because they do craze.

It might be well to consider the ceramic industry and its problem of crazing as typical of an industry which was unquestionably the first

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* A paper by the chairman of Committee 7-1, on Crazing, presented in lieu of a report of the committee, which has just been formed.

to encounter it and which has studied it and is still studying it so intensely that it is in a fair way towards solving its difficulties. Glazes were placed upon soft open porous ware not only to enhance it but to reduce its absorption and abrasion. The glazes in contrast to the ware were hard and not absorptive. In other words they were distinctly different and it was early recognized that this difference was the cause of the difficulty. As a consequence the plant expression for the cause of crazing of ceramic ware originated, and is indeed expressive—"fit" or "lack of fit"—the former indicates lack of crazing and the latter crazing. The industry recognized that if the coefficient of expansion of the unglazed ware and the glaze, and the modulus of elasticity of the two are too widely different then the glaze does not fit the ware and crazing must result. The same condition exists in the industries engaged in making cement products. If the surface does not fit the interior or body of the product crazing will result. The cause will again be the same, namely, differences in the coefficients of expansion with temperature changes, differences in the moduli of elasticity, and a third condition which does not exist in the pottery industry, a different coefficient of expansion due to absorption of water. We are purposely leaving out of this paper any discussion of strength. It is important and necessary in the question of crazing, but the strength consideration of cement products has been discussed so much to the total exclusion of all other considerations that to concentrate attention on the latter we will exclude strength in this discussion completely.

The simple statement of these considerations, however, does not assist much in solving the problem. Take for example the modulus of elasticity. We have become so accustomed to using the value of 3,000,000 for this coefficient that it is now almost axiomatic. But read what Walker has found and published in Bulletin No. 5 of the Structural Materials Research Laboratory, and note that the value is a function of the size of the aggregate as well as its grading, the consistency of the mix, the amount of the cement, and the age of the concrete, to which must be added the character of the cement and the character of the atmosphere in which aged, as to humidity, temperature, etc. The two other coefficients mentioned are affected in the same way. Hence, must not a study of the cause and prevention of crazing embrace a study of these three coefficients, and how the cement, the aggregate, the mixing water, the air, and the personal equation affect these?

Assuming that these premises are correct let us consider in more detail what we are "up against." First let us discuss the cement. I have already told the members of the committee that with myself as chairman it will always have in mind and carry out all tests with *cements* and not *a cement.* If it wishes to restrict itself to a cement or to an assumed average cement obtained by mixing representative brands it must operate under another chairman. Here we must assume that practically all portland cements are different, and mixing gives only an average

of that mixture. The average prepared tomorrow by mixing samples of the same brands will quite likely give different results. But what data do we have of the character outlined above as requisite? The answer is none. We must have values for these three coefficients of a variety of different cements, aged under different conditions, and with ranges of consistencies deviated very much from the so-called normal consistency. It must be borne in mind that on the trowelled surface of a floor or the skin coat on concrete we have very largely neat cement, and here crazing is invariably manifest. But this is enough for cement, for more could be written about what we do not know of this commodity than of what we know.

Next we have the aggregate to consider. Here there is more information available but far more needed. For instance what do we know about the coefficient of expansion of any concrete in a saturated atmosphere, in a 50 per cent saturated atmosphere, and in a very dry one? There are several well-accepted coefficients, but on examining the original articles in which these values appear we do not find any data which will enable us to reproduce these concretes, other than the simple ratios of cement to sand to large aggregate. These data were obtained by those who did not appreciate the importance of size or grading or volume of aggregates, or the differences between the various kinds of crushed stone, gravel, slag, etc. Apparently any 1: 2: 4 concrete was representative of all 1: 2: 4 concretes, so long as the cement was purchased as portland cement, the fine aggregate as sand, and the large aggregate anything graded between ½ in. and 3 in. regardless of whether it was uniformly graded or all one size—and no notice made of these facts. It has been shown and it is now an accepted fact that the quartz gravel is a poor aggregate to use as a fire resistant agent, due to the sudden increase in volume of quartz at a fairly moderate heat, as it changes from one form to another. Possibly other forms of aggregate have just as undesirable coefficients at atmospheric temperatures, even though there is no conversion from one form to another, but our information is lacking.

Water, the third item needing study, is one that has been so properly and effectively emphasized for the past ten or fifteen years that its true relation to concreting values, as regards strength and certain kinds of durability, is now recognized. We also have data showing how different amounts of mixing water affect the elasticity. We also in general believe that excess mixing water results in excess shrinkage or drying. But with these, our knowledge and our beliefs on the effect of too much or too little water largely cease. It is our pleasure generally to look to the other side of the street in deference to our enthusiasm in concrete as we pass, as pass we must on our way from the bureau to the business center of Washington, a very long series of exterior steps leading up to an imposing mansion. The owner of the latter is a builder and contractor and is the author of his own steps. If he would have purposely tried to demonstrate how excessively exterior stairs may craze he could not have suc-

ceeded more eminently. He has never inflicted a more unsightly job upon a customer than he has upon himself. We have often wondered if they were not made in the typical manner, that is, a very dry base rammed against wooden risers, and then finished with a rich mortar. Most of the hydration of the base has taken place by means of rain water penetrating the rich mortar, with the result of the marked expansion of the base and crazing. Incidentally, one of our committee members believes that crazing is due to expansion and not contraction, and we believe that he is right, as are also those who believe it is caused by contraction. But evidently both are not right at the same time. In any case do we know that water will not cause contraction as well as expansion, depending upon whether the optimum amount has not been reached or exceeded? We may feel unduly optimistic, but we feel sure that if the committee ever does real work and produces a worth-while report it will have made a valuable contribution to the question of volume changes in concrete, although it is specifically dealing only with linear changes on its exterior. Water,—both that used in mixing and that absorbed or given off in later life, is the big factor after the cement, in volume changes and in crazing.

The atmosphere is the next item that must be taken into account in any discussion of crazing. This has been recognized for a long period and is evidenced partly by the covering of concrete roads with various materials to prevent the rapid evaporation of the mixing water. But noting this the critical observer must ask why we do not logically treat concrete sidewalks in the same way. Some may say that the road does not craze but the sidewalk does. The answer to this argument is that the road does not show the crazing as prominently as the sidewalk, but it is not yet definitely proven that the saturated atmosphere is the ideal one for hardening of all concrete. If all the water used in making concrete and needed in mixing it were combined and so fixed in it as a result of the hardening phenomena that it would never be lost, I am thoroughly convinced that the concrete would be valueless, for it would be a soft product, in no way giving the service required of it. It would mean in general that the cement in concrete would have combined with it a weight of water from 50 to 100 per cent of its own weight. We do know that hydrated cement containing that amount of combined water has no bonding value. Consequently, since a very considerable part of the mixing water must be removed, when shall it be removed? Shall the concrete be kept saturated for a month and then allowed to dry as rapidly or as slowly as the atmospheric conditions permit? The answer is, candidly, not known. The proper atmosphere for the proper hardening of concrete is yet to be determined.

There is reserved for the final citation that large group of variables which are constant only in that they are constantly before us. One of the outstanding items in this group is the so-called personal equation, evidenced by that peculiar propensity of human nature to do a thing in one way and more or less believe and report that it has been done in

another. As an extreme instance we have the giving of data showing 10 or 12 per cent water used, whereas 17 to 20 per cent went into the job, or making a 1: 1½: 3 concrete according to specifications and a 1: 2: 4 according to bills rendered by the sand and gravel dealers. Then we have the more common cases of the lack of proper knowledge of materials introducing variables, such as a lack of taking into account the variable moisture in the aggregates, the lack of uniformity of grading, the lack of uniformity of mixing, etc. We also have a variation in the character of the sub-grade, the form, the molds or the backing, all of which introduce factors concerning which we know so little. Such a group of extremely difficult items to control does not tend to uniformity in production, but at the same time it furnishes conditions which must be studied in part at least in order to have available more readily enforced precautions as would tend to reduce the difficulties which are the subject of the work of this committee.

It might be inferred from portions of this discussion that the way to avoid crazing would be the doing away with any surfacing on concrete, and consequently no investigation would be needed. This is true in part, and could we always have the surface on the cement product identical with the rest of the piece we undoubtedly would not have crazing. Nevertheless, volume changes might produce a cracking which, however, would be distinct from crazing. But there are so many cases where a surface is intentionally or unintentionally produced that due consideration must be given to its cause and prevention.

This paper is presented especially to bring to the Institute the thought that crazing is no simple problem, to be solved by a field examination followed by a more or less perfunctory laboratory research, and climaxed by a voluminous, well-illustrated report, unanimously signed by the committee members. On the other hand it is a problem which will cost the active services of several trained investigators over a period of years, and in which a committee can act only in an advisory capacity, as a collator of data, and in giving a breadth of view to the discussion and the problems of the latter. The committee itself cannot solve the problems involved. No committee can. The problems are not of that type. The committee needs that kind of co-operation which is so difficult to obtain—financial—either direct contributions of funds or the services of investigators placed at its disposal. It will have a first meeting during this convention and formulate methods, procedure, and possibly a program. How successful it will be in obtaining this co-operation will be the subject of the first report of this committee.

DISCUSSION.

Thaddeus Merriman.—Last summer we carried on some investigations of this kind. It was our belief that crazing might to some extent be due to the solubles in the cement. The thought was that if washing was good for the sand and gravel it might also be a good thing for the cement. This was done by putting about six inches of cement in the bottom of a pail, filling it with water and after a vigorous stirring allowing the mixture to settle. Then the water and all of the fine laitance and loose soft material which settled on top of the heavier cement was siphoned off. The remaining cement was then mixed two to one, with sand of average quality and spread out in a circular slab eighteen inches in diameter and 1 in. thick. The slab was then allowed to remain in the hot sun and not thereafter moistened. No single crack of any kind developed in mortar so treated. An analysis of the salts that came off with the water in the washing process indicated 36 per cent of lime, 29 per cent of SO_3 and 16 per cent of the alkalis with 19 per cent of CO_2 and water of hydration. Mr. Merriman.

About four years ago we had much trouble with crazing, which occurred during the first few hours after the placing of concrete in the Gilboa dam. These crazings the morning after placing were anywhere from 30 to 60 in. long and from an eighth to five thirty-seconds of an inch wide and extended from four to six inches down into the mass. It was a most distressing condition, and we did not know just where the remedy lay, but it was finally discovered by beginning to wet the surface about two hours after placing, and by thereafter keeping it continually wet that all cracking and crazing was prevented. The wetting of the surface operated to stop the movement of the fluids within the mass upward to the surface where, by evaporation, they passed off, leaving behind the solubles which they had contained. The alkali concentration in the upper layers finally became so great that the set of the cement in those layers was very materially accelerated. As a result of this acceleration as well as on account of the deposit of the solubles in the outer layers, a large part of what we call crazing occurs. There is much crazing that is not visible to the eye for some time after placing. Later on, however, when moisture penetrates and temperature changes do their work, these originally incipient cracks increase in size and become visible. The lesson we learned from the magnified cracking was, after all, a good one, because it served to sharpen our wits as well as to indicate some, at least, of the causes which bring about crazing, both big and little.

J. W. Lowell.—Did you carry on any parallel tests to determine the effect of washing the cement? Did you carry on any tests where you left the pats you made to be cured in moisture or cured wet rather than just drying out, to determine whether or not any cracks developed in pats that were cured in moisture? Mr. Lowell.

Mr. Merriman.—The pats cured in moisture? Mr. Merriman.

Mr. Lowell.

MR. LOWELL.—I mean the washed cement?

Mr. Merriman.

MR. MERRIMAN.—No, there was none made because there seemed to be no necessity for making them. The slabs of washed cement did not crack. They lay in the hot sun, and dried out immediately. The mortar was the hardest and densest I have ever seen. The slab rang like a bell. Of course it is possible to wash cement so much that it will never set. Some cements can be washed more than others and some cements show a material increase in strength under this treatment. For instance, it is possible with a washed cement mixed when moulded with 62 per cent of water to get at the end of eight days as much tensile strength as with 25 per cent of water and unwashed cement.

The most important characteristic of any mortar or concrete is its tensile strength. Concrete always fails in tension. Not until we learn to build up its tensile resistance will the cause of concrete be materially advanced.

Mr. Turner.

C. A. P. TURNER.—In the manufacture of artificial stone, we had an extended experience with crazing. We found in sand mold work that crazing depended in a large measure on the aggregate used. When we quit using Minneapolis blue limestone and used only crushed granite and washed sand, our troubles were practically ended. The idea that if a dry concrete is used trouble with cracking, crazing and disintegration is eliminated will not bear investigation. About twenty-five years ago the average specification for portland concrete was that it was to be placed in layers about four to 6 in. thick, mixed dry and rammed or tamped. Much work has been executed in that way with good aggregate which has not stood up well, which shows the marks of various layers, which has disintegrated more or less under frost action and shown both crazing and cracking.

A reasonably plastic mix gives better results and stronger concrete than a dry mix because as ordinarily placed much of the excess water leaks out through cracks in the forms and a certain moderate excess of plasticity helps thereby in securing a dense concrete by leakage and the settlement of the materials together in this way. In the laboratory test on the contrary where the water in the molds is prevented by paraffine from oozing out the strength is lessened and in like manner in placing the floor finish on a rough slab of concrete because the water cannot get away the floor finish will not stand well under trucking unless the water is limited to the minimum amount consistent to bringing the water to the surface when the finish is worked.

The lessons of the laboratory are valuable in substantiating practical experience in this respect but they have been to some extent misinterpreted in the ordinary run of rough work where excess moisture is eliminated as noted and there it is needed for plasticity in surrounding the steel and getting sound concrete.

The idea that we are going to design concrete mixtures by algebraic formulas does not impress one favorably. At a convention in Milwaukee

eighteen years ago this matter was the subject of extended discussion. Proportioning by weight to get the greatest density was the theme and on that basis seemingly galena as aggregate would secure the strongest concrete because that would be most dense from the weight value standpoint. The idea that the proportioning by weight must be varied with the weight volume of the different aggregate was only slightly touched upon and the method was open to the criticism of inferiority to the older idea of determining the percentage of voids in the coarse aggregate by noting the relative volume of water which a struck measure of the aggregate would contain. This method may not be applied to sand because of its fineness and we may have recourse to the extensive experiments by Feret and the diagrams showing the values of the variation in the size of the grains as a practical guide from our analysis of the sand by the sieve test.

In other words, on the theory that concrete is an artificial conglomerate stone the coarse aggregate may be viewed as a space filler and the mortar the cement that fills the voids and holds the coarse material together. On that simple conception we analyze the coarse aggregate for the volume of voids by filling it with water and then use the necessary amount of mortar not only to fill that volume of voids but allow a twenty to 25 per cent excess, thus providing for solidity and working plasticity.

LESLIE H. ALLEN.—Mr. Bates will get into difficulty because, if I understand him rightly, he says that if the crazing cannot be observed with the naked eye, he is going to use a microscope, and I suppose if a low-powered microscope won't show it, he will use a high-powered one and find the crazes somehow. I think the Institute ought to take his weapon away from him and rule that crazing is that which can be discerned with the naked eye. I believe that with that limitation, we can claim that in the product we make we do not get crazing. Just why, I do not know, because, according to all the theories, we ought to get it. We make a thin concrete product—a roofing tile. The body of the tile is a tamped concrete made relatively dry, of a one to three mix of cement and a fairly coarse, well graded sand. On top of that we placed a sixteenth of an inch of a very wet, 1:1 concrete with an admixture of about 25 per cent of either ferric oxide or chromium oxide, to get color. That is very wet and flows on the surface of the tile and is smoothed off with a trowel just once, and as far as I can see, we do not get crazing on that product. The curing is done at room temperature with a little sprinkling the first 48 hours and after that it is exposed to outside temperature, although protected from the sun. The problem of crazing varies, however, so much in different kinds of products, monolithic concrete work, stucco, wet-poured concrete products and dry-tamped concrete products that I presume there must be very different reasons for the phenomena of crazing in those different lines. I have been told by some people that perhaps the metallic oxide admixtures we used are what saved the day in our case. I would like to suggest to the committee that they have a look at the products we are making and see if their team eyesight, unaided, will observe any crazing. Mr. Allen.

Shall Anything Be Added to Portland Cement?

By Maximilian Toch.*

Portland cement manufacturers as well as the International Society for Testing Materials have been against the addition of any material to portland cement, and before the World War, the International Society issued a statement that nothing beyond 2 per cent should ever be added to portland cement. This statement needs qualification, for the gray, somber, monotonous appearance of many portland cement structures can be much relieved either by proper painting, or by the addition of the correct pigments.

This has led our laboratory to make exhaustive experiments covering a period of years, on the correct pigments which should be added to portland cement, for the ideal pigment would always be one that would not interfere with the setting nor with its tensile strength.

Several years ago, in attempting to devise a reasonably-priced blue-green, and a reasonably priced green, I found, what would most naturally appeal to any chemist, that either a carbonate of copper or some other insoluble salt of copper would be the logical material to use for making a bluish-green. Much to my surprise, a block made with various copper salts had about the tensile strength of a slice of rye bread two or three days old, and for a long time I could not account for this.

Eventually, and through further experimentation, it dawned upon me that no color is useful in portland cement which combines with lime, for it is generally known that anything which abstracts lime or combines with lime prevents the cementitious quality of the cement itself, and that is why copper pigments must not be used.

What really takes place is, that instead of the lime liberating and crystallizing thereby forming a reticulated structure, which bonds the small particles of aggregate together, the lime is transferred into a copper lime compound with disastrous results to the ultimate strength of the concrete structure.

The point I want to make particularly is, that many portland cement manufacturers do not hesitate to recommend as much as 9 per cent of a

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*Vice-President, Toch Bros., New York City.

pigment the composition and effect of which they are totally ignorant, yet they will not permit the addition of 1 or 2 per cent of a useful material which improves the concrete.

Twenty-five years ago a Belgian engineer, named DeMan, came to this country and started the manufacture of portland cement tile in various colors and in various designs, and it was then that I first noted that some of the pigments really waterproofed and gave additional strength to portland cement and some detracted. I was conducting some experiments for him at the time, on integral waterproofing, and I noticed that much more than 2 per cent of various materials might be added to cement which would decorate and at the same time increase the physical qualities of portland cement.

A general rule may be laid down, that a pigment that does not combine with lime, while it is being generated, produces in itself a tensile strength which is quite remarkable, the only unfortunate part being, as in the case of the 122 Dutch Blue (see Table II) which gives an enormous increase in tensile strength could not be used either as a waterproofing or wearproofing material, because the resulting concrete has a decidedly blue tone. Upon investigation these briquettes show that an additional amount of an aluminate has probably been formed which in itself is a cementitious material.

The lists which I append show the addition of 2, 5 and 10 per cent of various pigments, and also show quite conclusively that in the case of a chemically pure black, like No. 112 and No. 111, 2 per cent is all that is necessary, but 5 per cent will do no harm.

With the 113 B Pure Gray, which is a very dark gray, the increase in tensile strength is due to the addition of calcium phosphate, but both the 97 yellow and 127 green are not to be recommended on account of their abnormal reduction, and for the further reason that the pigmented concrete is soluble in water. *A pigment added to portland cement must not be soluble.*

Chemical engineers and concrete engineers have never co-operated as they should. For some reason, the concrete engineer while he is a mechanical or civil engineer has not regarded portland cement and its reactions in any other light than that of a physical cementitious material which needs little or no explanation. As a matter of fact it is a chemical compound and its reactions are purely chemical. No one understands this better than the manufacturing chemist who has built of concrete, and in my experience I have never seen a single instance wherein concrete in a chemical works can be depended upon, or can be utilized without either the addition of an integral compound, or the application of a coating material to preserve it against acid or alkaline decomposition.

There is a general consensus of opinion that concrete properly made needs no integral or surface treatment for waterproofing or hardening. This is perfectly true as far as it goes, but it does not go far enough, as there are many cases where the treatment of concrete even before it is finished, or after, is essential.

Take the case of concrete roadways. As soon as these are open to the public some sections begin to go to pieces at once. The new concrete road between Jamaica and Lawrence, L. I., which was completed a little over two years ago has been shut down occasionally for repairs, all of which could have been obviated if the road had been properly reinforced, and if the surface had been treated so as to prevent frost cracks, and surface disintegration.

Personally, I do not know of a single concrete floor in the interior of any building which is considered finished without hardening, painting, or integral material added, and the argument in the case simply sums up into the following: that you may be able to make concrete which is permanent, without the addition of anything, but the chances are very much against you. All the arguments of the cement manufacturer about the impermeability of portland cement and its lasting qualities under all conditions fall to the ground when it comes to the use of portland cement in chemical engineering work.

Bins for the storage of nitre cake, which is sulphate of soda containing about 30 per cent of free sulphuric acid, are exceedingly difficult to build of any material. The result is that most of it is stored in the open where the rain, snow and dirt attack it, and frequently render large parts of it useless. When the pile has been cleared away the earth is so soaked with acid that it destroys your shoes when you walk over it.

Eight years ago, when we went into the World War, it became essential to construct a building where thousands of tons of this material had to be stored and used for the manufacture of material essential in the conduct of the war. It was utterly impossible at that time to obtain sufficient acid-proof brick or porcelain tile, or to find a perfectly acid-proof binding material, or to obtain the necessary skilled labor to erect a storage building of this kind. Even had all these things been obtainable the cost would have been prohibitive; so, much to the chagrin of my associates, I decided to build a concrete warehouse which was nothing more or less than a pit—50 ft. x 150 ft. and 10 ft. deep, with a superstructure of wood. This pit was built entirely of concrete, properly reinforced with reinforcing rods coated with an acid-proof material. The entire mass contained an acid and waterproof integral mixture, and the surface was ¾ in. of 1: 3 concrete composed of 1 part portland cement, 2 parts of sand and 1 part of the same acid-proof integral material troweled down hard and dry. After ten days the surface was again treated with an acid-proof filler and a coat of paint. This was in 1917. The last time I examined this building carefully was in December, 1924. It had been continuously used for the storage of nitre cake, and it was still in perfect condition. No leaks had developed, no surface erosion appeared, because the free sulphuric acid had had no chance to attack the concrete. Now, I ask, in all fairness, whether a concrete bin could be built of only portland cement, sand and aggregate, which would have stood more than a few months.

I know of many other cases of mechanical works, where portland cement

could be used, if properly treated, but where attempts to use it have been failures because untreated portland cement is neither acid-proof nor alkali-proof.

I think it will be admitted that concrete will not stand in an alkaline soil, and no perfect integral or surface treatment has yet been worked out which gives complete satisfaction for a number of years.

Admitted, that there are many cases where portland cement must be treated either during or after its construction, there is no one panacea which covers it all.

The "black eye" that the treatment of portland cement has received is unfortunately due to some of the poor materials that have been exploited and that have failed.

THE CORROSION OF STEEL IN CONCRETE.*

When steel corrodes its volume increases in relation to its molecular weight, and this increase is as 112 is to 211. This, of course, varies with the nature of the corrosion, and I am taking yellow rust as a standard.

As far back as 1907 when I examined the corrosion of the steel columns on the New York subway, and later in 1912 I examined the corrosion of the steel of the Battleship "Maine," which had been lying in and out of sea water for thirteen years, I made the fact public that rust was not definitely the tri-hydroxide of iron $Fe_2O_3\ 3H_2O$, but it could be a magnetic form of black, and it could also be the brown variety containing less water than the yellow. What I want to point out particularly is that any increase in volume produces pressure so that if we have a film of paint and rust starts underneath the increase in volume of the rust as compared with the steel itself lifts off the film of paint, for it is a well-known law in physical chemistry that any reaction that produces pressure is retarded by pressure. For instance, if you take a bottle and put in a few pieces of zinc and hydrochloric acid, effervescence takes place, hydrogen is liberated and chloride of zinc is formed, but if you take a strong bottle and hammer a cork down tightly into it, the chemical reaction stops because the pressure of the walls of the corked bottle is greater than the pressure generated.

Tradition has taught us that concrete prevents corrosion of steel, and progress is retarded on account of tradition in many instances. If a steel bar is embedded in concrete sufficiently deep and kept back from the exterior face sufficiently far so that the weight of the concrete is greater than the pressure produced by corrosion, you will have no corrosion, but if the bar is sufficiently near the surface so that the action of water and air can reach the steel, you not only have corrosion, but the pressure produced by it splits off the surface of the concrete. These reactions are plainly seen in the Figs. 2, 3, 4 and 5. The general statement, that concrete prevents

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* It gives me a great deal of gratification to acknowledge the co-operation of T. Arthur Smith, vice-president of the Turner Construction Co. for loaning me some of the photographs which appear in this illustration.

corrosion is not correct, in fact, concrete may accelerate corrosion if the amount of lime liberated is below a given strength.*

Take the case of a steel drum in which ammonia is shipped. Just as long as the ammonia is of sufficient strength no corrosion takes place on the inside of the drum because the alkali inhibits it, but empty the drum and fill it with water, or reduce the ammonia below a given strength, and you produce corrosion even though the liquid is strongly alkaline. If you take portland cement and mix it with two parts of sand, and then embed a piece of bright, clean steel in it ¼ in. below the surface, you can

FIG. 1.—REINFORCED-CONCRETE ROOF IN CHEMICAL WORKS WHICH COLLAPSED OWING TO THE CORROSION OF THE STEEL THROUGH THE CONCRETE.

The roof was made in slabs 4 in. thick, one part of cement, 3 parts of sand.

submerge this experiment in all kinds of corrosive liquids, and no corrosion of the steel takes place, for two reasons, first, because a 1 : 2 mixture sufficiently trowelled is impermeable, and secondly, because the amount of alkali generated by a rich mixture of that kind is sufficiently great to prevent corrosion. But if you take a steel bar embedded in a 1 : 2½ : 5 mixture you will have corrosion, or you will prevent corrosion depending upon the distance that the steel is from the surface.

*Corrosion of Iron and Steel; Dilute Alkaline Solutions, J. Newton Friend; See also—Heyn & Bauer; Cribb & Arnauld.

A number of my engineering friends have taken issue with me that reinforced-concrete construction is a civil and mechanical engineering problem, and not a chemical engineering problem. I differ from them in so far as I believe reinforced-concrete construction is mechanical, civil, and chemical, and that it is by far wiser for the three types of engineers to get together and co-operate than it is for one of them to resent the advice of the chemical engineer. I am sure we all want to make progress. My personal attitude has always been that a blind hen occasionally finds a grain of corn, so that even we chemists may stumble upon facts which will help mechanical and civil engineers in proper construction, so let us see how best we can get together in order to conserve concrete of all types.

I have been hampered very much by the fact that I have been making materials for the preservation of concrete for 30 years, and every time I suggest something of the kind the commercial aspect obtrudes itself more or less. Therefore in this article it must be patent why there are many cases where a non-proprietary article will do. An engineer could paint reinforcing rods with a mixture of neat portland cement and lime water so as to be sure that the percentage of alkalinity generated is beyond the corrosive point, and in order to frost-proof the surface, he could buy any one of the good mixtures on the market composed of zinc and magnesium fluosilicate which would precipitate enough silica into the pores of the concrete to prevent moisture getting through.

Every engineer, of course, understands that there are two elements that produce rust: one is air (oxygen), and the other is water, but pure air that contains no water is non-corrosive, and distilled water or boiled water that contains no air is non-corrosive. A piece of steel exposed to the air at New Mexico or Arizona remains bright for many months because the air does not contain sufficient moisture to produce rust. I believe engineers will agree with me that the proper place for reinforcing rods or bars to obtain the greatest compressive strength should be near the surface. Unless the surface be waterproofed, or the bars treated so that they would not corrode, the same effect will be produced as is shown in Figs. 2, 3, 4 and 5.

If cement wash, as an exterior finish, is an added protection on a concrete building, how much more of an added protection would an acid resin paint be with a waterproof coating, but the only trouble with a cement wash is, that it may or may not set, and if it does not you will simply have a coating of dust.

The whole problem of the preservation of steel, whether it be reinforcing bars, or whether it be a bridge, is not quite as difficult as most engineers seem to think, for co-operation with the chemical engineer so that he may obtain at least a little share of the credit, is going to improve conditions all round.

One of our great bars to progress in concrete has been tradition. I take it that every one present remembers how years ago tradition had it that concrete would not stand sea water. Up to 1912 there was no con-

FIGS. 2 TO 5.—CORROSION OF THE STEEL WHICH PRODUCED PRESSURE AND SPLIT OFF THE SURFACE OF THE CONCRETE.

This would not have happened if the steel had been painted or if the exterior of the concrete had been coated to prevent the access of water.

crete sea wall built by the United States Navy that did not look as if some giant monster had bitten big pieces out of the structures. There have been many theories advanced as to why concrete will not hold in sea water, but the simplest one and the most tenable one is the solubility of lime in pure water and its greater solubility in salt water. The argument that no material need be added to portland cement to make it absolutely perfect provided enough cement is added, and the mixture properly made, has never convinced me.

When the United States Navy built its first great dry dock, known as Dry Dock No. 4 in the Brooklyn Navy Yard, the chief engineer determined to waterproof every part of it excepting the bottom, and the bottom being many feet thick and carefully laid, has cost thousands of dollars to repair, and even today is not in anything like good condition. Yet the side walls which were waterproofed are all perfect, and so the other four great dry docks built by the Navy have been waterproofed, and in not a single instance has the slightest deterioration taken place. If this argument will not hold, I would very much like to have it answered.

Portland cement has been standardized for many years. I do not doubt for a single moment that this has worked against its universal use rather than for it. No portland cement manufacturer has dared to improve his product by altering it from a prescribed composition. The holy circle drawn around it, has included, that it should set within a prescribed time; it shall have a given tensile strength; and nothing shall be detracted from it, nor shall anything be added to it, and the result has been that it has stood still.

It is perfectly possible to make portland cement set quicker than it does without detracting from its physical qualities, yet such a procedure is tabooed.

In 1918 the Germans built a gun which had a range of 74 miles. So secretly was this done that the foundation on which it rested was built in 24 hours, and destroyed in the same time. It is now well known that that foundation was an oxychloride cement and not a portland cement. New stuccos which set quickly, and have been giving excellent satisfaction are made both of oxychloride and calcium sulphate quick setting cements.

Lately there has come on the market a cement the probable composition of which is a calcium aluminate and calcium ferrite (lumnite) which in 24 hours attains 91 per cent of its ultimate strength. There seems to be a great future for this cement.

There are many good materials on the market which should be used for coloring cement, waterproofing cement, lubricating concrete, and protecting it against acid and alkaline conditions which normally destroy concrete, and the same liberal interpretation for the use of added materials to increase the utility of concrete and concrete structures is very much to be desired.

CONCLUSION.

Ordinary Venetian Red such as is indicated by 179 Terra Cotta or any red oxide containing calcium sulphate is dangerous as the mortar color. Chrome yellows and copper greens similar to 97 and 127 are not to be used as they detract from the lime.

It is more important to paint reinforcing rods either with an alkali-proof paint or with a wash of pure portland cement than it is to let them go uncoated. The popular fallacy that rusty iron adheres to concrete is an error as is shown by the photographs in the illustrations.

Where calcium chloride is used as an anti-freeze or some other of the basic chlorides, they are very likely to rust steel in spite of the excess of lime that may be in the concrete. It is therefore essential to paint them with an alkali- or chloride-proof paint, and in that case, a simple wash with concrete is not sufficient.

97 Yellow.........Basic zinc chromate
163 Oak...........Ferric hydroxide and colloidal aluminum silicate
165 Light Drab.....Ferroso ferric oxide aluminum silicate
127..............Copper carbonate
505 Willow Green...Chromium oxide, free from soluble salts
535 Sea Green......Mixtures of 505, 165 and carbon black
715..............Meta para ortho nitro toluidine
465..............Paramit ranaline precipitated barium sulphate and alizarine
713 Spruce.........Mixtures of 179 and 163
179..............Calcium sulphate, ferric oxide and clay (Venetian red)
137..............Ferric oxide and colloidal silicate
429..............Ferric oxide
106..............Ferric oxide, ferrous oxide, and manganese hydroxide
113..............Calcium phosphate and carbon
94..............Principally carbon
122..............Cobalt blue, free from soluble salts
520..............Oxide of iron
116 Black..........Oxide of iron and manganese
111, 112 ....Black carbon

TABLE I.—TENSILE STRENGTH: 1:3 MORTAR WITH COLORING PIGMENTS ADDED

	No. 97 Yellow	No. 163 Oak	No. 165 Light Drab	No. 127 Green	No. 505 Willow Green	No. 535 Sea Green	No. 715 Fire Red	No. 465 Ox Blood	No. 713 Spruce	No. 179 Terra Cotta	No. 137 Spanish Tile	No. 429 Cherry Red	No. 106 Light Brown	No. 113B Pure Grey	No. 94 Dark Slate	No. 122 Dutch Blue	No. 520 Dutch Tile	No. 116 Black	No. 111 Black	No. 111 Black	No. 112 Black	No. 112 Black	Atlas White used in test
Per cent color added.	10	10	10	10	10	10	5	10	10	10	10	10	10	10	10	10	10	10	2	5	2	5	..
7 days...	95	285	285	75	245	265	285	385	280	80	70	325	295	320	330	505	335	280	295	340	240	180	290
	95	295	300	85	250	270	300	460	280	80	70	330	310	320	355	505	340	295	305	340	250	195	295
	100	320	310	90	275	290	330	465	295	95	75	360	350	345	360	545	350	305	310	360	285	200	330
Average....	97	300	298	83	257	275	305	437	285	85	72	338	318	328	348	518	342	293	303	347	258	192	305
14 days...	.	400	380	170	325	365	385	345	390	80	360	415	425	420	415	550	410	380	380	380	330	265	420
	160	410	390	180	345	380	410	350	400	85	360	435	445	445	425	580	435	385	400	400	335	265	425
	180	420	400	185	350	395	410	365	400	85	365	460	450	450	450	590	440	400	400	410	370	270	430
Average....	170	410	390	178	340	380	402	353	397	83	362	437	440	438	430	573	428	388	393	397	345	267	425
28 days...	...	435	460	290	375	400	425	420	425	120	480	440	465	510	445	625	470	450	445	415	360	310	400
	320	455	465	300	410	420	430	425	450	120	515	465	475	515	445	625	470	450	450	420	370	310	450
	325	465	485	310	435	440	450	425	480	130	520	490	490	530	480	655	520	455	480	450	380	310	455
Average....	323	452	470	300	407	420	435	423	452	123	505	465	477	518	457	635	487	452	458	428	370	310	435

TABLE II.—TENSILE STRENGTH OF CEMENT CONTAINING 10 PER CENT OF COLORING INGREDIENT

Blank (10 per cent Sand)	No. 122 Blue	No. 179 R Venetian Red	No. 35 Bright Yellow
386	447	108	365
358	508	96	395
...	...	112	366
...	...	93	358
372	478	102	371

(1) No. 122 Blue increases tensile strength of cement.
(2) No. 179 Venetian Red decreases tensile strength of cement very appreciably.

No. 179

$CaSO_4$	35.70 per cent
Fe_2O_3	39.75 per cent
Inert Base (Clay)	24.55 per cent

The analysis of No. 179 (Venetian Red) such as is commonly used for red coloring in mortar and concrete.

TABLE III.—TENSILE STRENGTH OF 1:3 MORTAR CONTAINING 10 PER CENT OF COLORING PIGMENTS

Color	No. 429 Cherry Red	No. 465 Ox Blood	No. 94 Black	Blank
Breaking strength, 28 days	321	328	374	308
	351	333	418*	316
	441*	361*	375	274
	379	320	380	315
	341	329	366	343
	386	322	393	345
Averages	370	(5) 326	(5) 377	317

DISCUSSION.

Prof. Hatt. PROF. W. K. HATT.—Perhaps Dr. Toch can answer a question which is troubling me. He has told us that the prevention of rusting of steel is affected by the weight of the concrete acting against the expansion due to rusting and, therefore, the steel should be as far from the surface as possible. Now a heavy mass of wet concrete when exposed to a rapid drying atmosphere shrinks on the surface and cracks or crazes are developed, thereby permitting rust producing agencies. It the steel is put near the surface, these cracks are to a substantial degree prevented. Some middle position between the surface steel and steel too far embedded should be determined.

Mr. Toch. MAXIMILIAN TOCH.—I said you could put the steel near the surface provided you coated it with an alkali proof material. Rust is produced through water and air; one of them alone will not do it, so if you keep air and water out of the concrete, you can keep rust away from the steel. The reaction that takes place in rusting is one of pressure; that is to say, 112 lb., that is the atomic weight of two iron, produces 221 lb. of rust, so you have double the bulk there. Now if you have a heavy enough weight on top of that, the oxide of iron cannot form, and consequently it cannot lift if it does form, so it stops, because any reaction—this is a fundamental law in chemical physics—that produces pressure is retarded by pressure, so no matter if you did have a crack and put muriatic acid down that crack up to the steel, if you had enough weight on that steel, no rust would take place, except at the point of contact.

Mr. Wight. FRANK C. WIGHT.—I do not follow Dr. Toch's reasoning on the performance of the under side of a reinforced-concrete beam. Certainly there is no weight on the two, three or four inches of concrete which lies beneath the reinforcement in the lower flange of such a beam.

I probably have been unfortunate enough to have seen as many concrete buildings which have failed as anybody in this country. In a concrete building failure, you can see the performance of steel in concrete better than you can in any other way. It is very rare, in my experience, when you examine a concrete failure, to find any rust evidence on the steel, or on the concrete which has been lifted off of the steel, and still bears the impress of the curve of the rod. It is a fact that in practically all concrete failures, particularly where you get a failure of a slab with a large number of rods and a wide plane, that the impact of a falling mass—and this failure I am commenting on is the failure of a fairly good concrete resulting from the impact of a falling mass which was not so good—the falling mass practically invariably takes all the concrete from the rod. In comments on concrete failures I have frequently seen it stated that the

concrete could not have been good and the bond between the concrete and the steel could not be good because all the concrete was stripped from the rod. I have never seen a failure where a floor slab collapsed and hung down, where the concrete did not strip from the rod. Such behavior is no indication of the condition of the concrete. In such cases there is rarely evidence of incrustation of rust on the concrete that remains and very little evidence of rust on the steel itself.

It seems to me that that tendency to rust is due to the lack of density of the concrete and not to the lack of weight.

C. A. P. TURNER.—I was called some years ago to investigate a concrete bakery building. The floors were all heavy, having furnaces 150 tons weight on some panels. The complaint was that the concrete was getting out of shape so much that a hundred dollars worth of glass per month was broken in the windows. I could not find anything the matter with the concrete. The steel sash were bulging out of line and the glass was cracking. I found that the steel was rusted from steam and fumes to the extent that a layer of rust from an eighth to a quarter inch had formed under the base and top of the sash causing the muntins to bow out of line an inch or an inch and a half and in like manner the rust between the glass and the muntin by expansion was squeezing the glass and causing it to break. I advised the owners to put in wood sash or secure stainless steel sash in place and they would then have no complaint about the concrete on this ground. **Mr. Turner.**

P. H. BATES.—In the first place, I am glad Mr. Turner digressed a little on this question of the corrosion of steel, largely because Mr. Toch, as I understand him to say, had said that the question of corrosion of steel was settled seventy-five years ago. If it was, it is unfortunate that the steel manufacturers have never taken advantage of that, because we still have corrosion of steel. **Mr. Bates.**

Dr. Toch refers to the standardization of cement, or, as I would get it, the over-standardization, and while he hands us a little bit of Irish confetti along that line, I feel that his last page comes to us as a real award of merit, because I notice he refers particularly to the undesirability of too much calcium sulphate. Now the committees that have been working on the specifications for cement, have put a maximum limit already on the SO_3 contents of cement as you buy it, so that while we may have erred somewhat in our standards for portland cement, I think Dr. Toch will agree that we have not over-standardized along this particular line. I do not belong to any one of the class of professions which may be referred to as hyphenated,—I am not a civil, mechanical, tonsorial or any other kind of engineer. I am only a simple chemist and consequently can take exception to Dr. Toch's statement that the chemical engineer is the man who is going to settle this question for us. I think the party or parties who settle this whole question of the durability and proper use of concrete is not going to be any type of engineer; on the contrary, he is going to be a physical chemist. As engineers you are too much engineers and not

enough chemists. Although engineers are presumably applied physicists, unfortunately in applying their physics, (I can say this, not being one of them), they have forgotten most of it. The physical chemist is neither a chemist nor a physicist, but he has retained a few of the fundamentals which he learned, and applies physics and chemistry in the solution of his problems. I think possibly one of the reasons why we are in the present status in regard to our knowledge of cement and concrete is that the chemical engineer has had too much to do with it and the man who should have been working on it, is this man who will apply the fundamentals of physics and chemistry to reactions that are a combination of those two and have not anything at all to do with engineering. The so-called engineer, or the real engineer (and you are all engineers here, or the majority of you are), is of that type who has forgotten most of his physics. That is unfortunate. Had you retained as much of your physics as the physical chemist has, we would have been further along. As a matter of fact, in this question of the solution of cement and what it is and what it does—all the progress we have made to date has been made by physical chemists. There has been practically no progress made by the pure chemist and we cannot hope from them in the future. The chemical engineer has been at it for a good long time, and judging from the nature of the discussion we have had here tonight, we are many miles away from an answer.

Dr. Toch.

DR. TOCH.—I would like to reply to Mr. Bates: In the first place, I did not say that the problem of corrosion had been settled; I said the theory was settled seventy-five years ago by Ditmar and I was referring to Ditmar's Anorganische Chemie written about 1854, who, long before any of us here was born, gave a full and complete account of the theory of carbon dioxide corrosion, and in 1902 there appeared that monumental paper by Dr. Whitney, head of the General Electric Co., who demonstrated the ionic theory of corrosion and then after that appeared endless papers showing the electrolytic theory, so that at least we know what corrosion is. I did not say that the matter had been solved in a preventive way; that is entirely another question.

Mr. Turner.

C. A. P. TURNER.—In coloring concrete we can learn from the methods of the Cubans in their manufacture of Spanish tile. On a trip to Havana I visited three large factories making tile 8 in. by 8 in. square by 1¼ in. thick of all colors. Some of these floors have been in use for forty years of constant wear without dusting. The colors used were mineral colors only. That is natural mineral aggregate. Their work is successful. There is no dusting because the tile are really hardened by mechanical pressure. The facing is neat cement and color mixed to the consistency of plaster of paris. The backing is a dry mix of sand and cement. The sand being heated or baked so there is not a particle of moisture in it. The different colors are separated by a steel scroll of band iron forming the pattern and presenting pockets when the scroll is placed on the steel face plate of the machine. These pockets are filled with a scoop for about ⅝ in. Then the scroll is removed, neat cement is dusted on the surface and the balance of

the die is filled with the dry mixture and struck off level. Then it is pressed with 5,000 lb. per square inch pressure, and instantly the block is hardened by forcing the moisture from the downward face up. The tile are dried in a rack for thirty days without sprinkling or other attention, when they are ready to ship.

Mr. Allen.

LESLIE H. ALLEN.—This matter of coloring pigments is, I believe, a matter of far greater importance to the future of the industry than is perhaps realized, because if we can get the polychrome effects in concrete, either monolithic or precast, that have been obtained in terra cotta construction, it will open up a tremendously wide field for the further use of concrete in decorative work. In this matter of pigment, I believe that the problem is not entirely a chemical one. It is true that a pigment which is partly soluble in water is unsatisfactory. The few experiments I have been able to make in the plant seemed to indicate that the specific gravity has a good deal to do with it, too. We have used ferric oxide and chromium oxide and ultra marine blue, all of which had a specific gravity fairly near that of cement. When we came to use lighter pigments, such as lamp black or heavier pigments such as compounds of antimony and lead, we did not get satisfactory results; in some way or other they would not combine with the cement we were using and the results were quite unsatisfactory. That seems to lead me to the same conclusion that Mr. Bates reached, that the physicist has quite a little to do with this. I got very little help from the consulting chemist I went to and it may be that if I consult Mr. Bates on our problems, I will get to the bottom of all of them.

Proportioning Concrete Materials With Especial Reference to Highway Construction.

G. W. Hutchinson.*

Few data are available relating to the development of concrete, as a unit, for particular types of structures. From the indications of this paper, it appears logical and economic to depart from the universal application of single theory proportioning of concrete for all purposes. Greater economy may be secured by considering the type of structure when designing the concrete mixture.

The data given bear directly on the design of concrete mixtures for highway construction. They appear to be more or less contradictory to present tendency or practice, with reference to either building or highway work. The economic application to the former may be questioned except upon large scale operations. It is felt that application to the latter should be given serious consideration, especially with the growing tendency to use the larger sizes of coarse aggregate, and the great amount of segregation taking place during handling from cars to storage piles.

In general building and mass concrete, other than highway construction, certain safety factors are included in design, and loads approaching the ultimate are in force at early stages after the completion of the structure. The period at which the structure is called upon to absorb loads nearing the ultimate is earlier than in highway construction. The pavement is not allowed open to traffic until it is about a month old, and at first bears applications of lighter stresses, both with reference to numbers and weights. The new artery offers additional inducement to both commerce and pleasure, both of which serve to stress the pavement to increasing amounts. A glance at Fig. 1 gives logic to an assumption that highest ultimate strength, with consistent early strength, is the principal feature to be considered in the design of concrete mixtures for this type of construction.

In the design of such mixtures, there is opportunity for considerable investigation. This applies not only to the theoretical proportioning of cement and different sizes of aggregate, but to an adjustment of such theories with respect to the quality of local aggregates most economically available. It also applies to such changes in relative strength which take place in concrete as the actual strength increases.

*Eastern Manager, Concrete Dept., Celite Products Co., Chicago.

Other features to be accorded consideration will be the stresses predominant in the pavement, and proper methods of testing to allow direct application of the results to field practice.

The method of proportioning concrete materials and the type of test to be used are most important in making laboratory work of practical

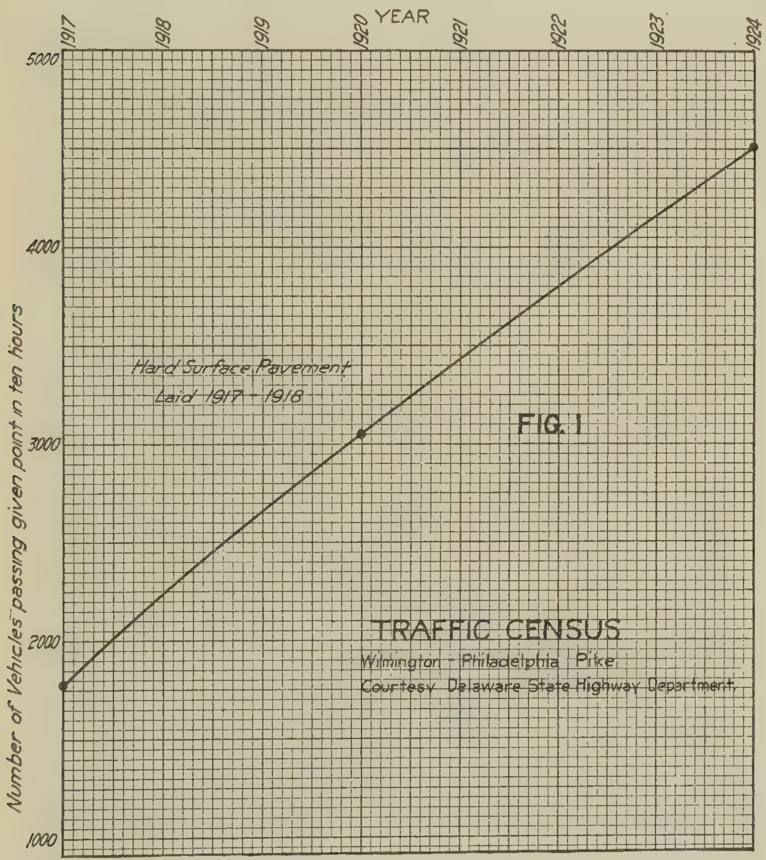


FIG. 1.—TRAFFIC CENSUS ON WILMINGTON-PHILADELPHIA PIKE.

value. Sufficient inconsistency exists between theory and practice to make it evident that new methods of proportioning aggregates for field concrete will be forthcoming before economic progress can be made. The arbitrary method of proportioning is acknowledged to be crude, but is yet to be replaced by more scientific methods. While improvement is being stimulated locally, general encouragement is lacking, and sometimes opposition is met.

Under present methods of proportioning, errors or inaccuracies, even within the limits considered acceptable by the method used, not only affect the single material in which such error is made, but change practically every ratio existing between the different ingredients of the mixture. When the total effect of such changes is considered, it is not to be questioned that reason exists for the variation encountered in field concrete.

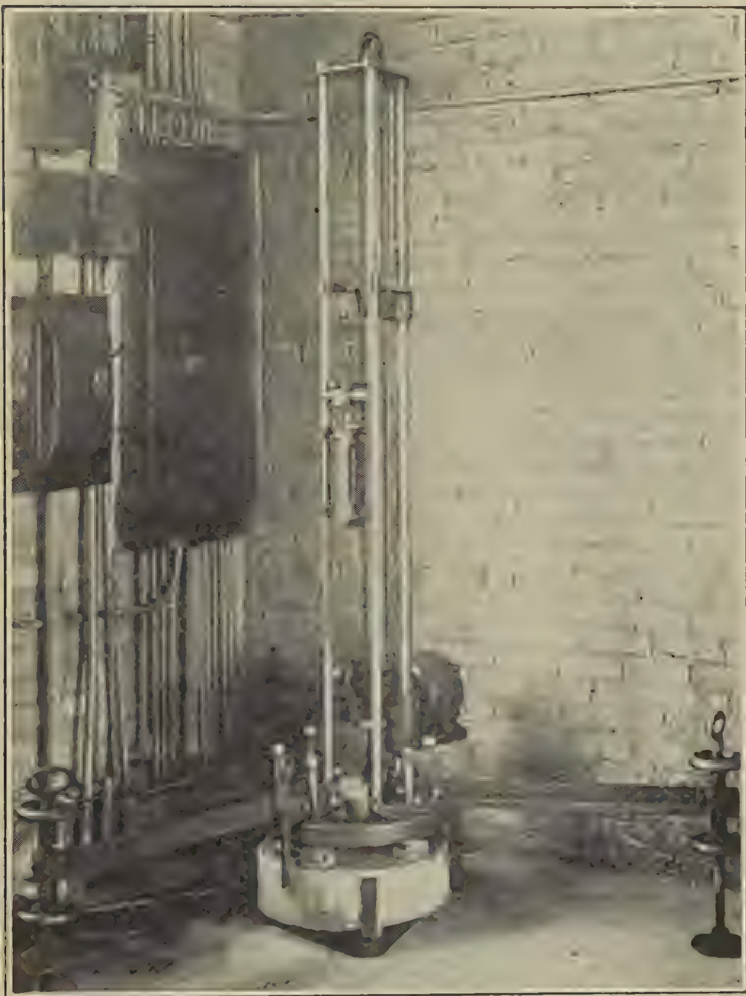
This paper brings out a new method for proportioning concrete materials and the data derived from it, given in summarized form, appear to point out definite facts. The method was designed to represent conditions existing in concrete after it has hardened in place. The tests extend up to a period of one year. The summary, from which the curves are platted, includes several thousand compressive tests and several hundred impact tests of concrete.

The compressive tests consisted of 6 in. x 12-in. cylinders. The impact test consisted of a 10-lb. weight falling from a constant height to a steel ball, 3 in. in diameter and resting in the middle of the top of the specimen. The specimen was 6 in. thick and 20 in. in diameter. It was supported on three-point rubber padded bearings. Each bottom bearing was connected with two rubber padded clamps on top, giving three sets of three-point bearings vertically to govern rebound. The weight used, the height of drop, and the size of specimen are all arbitrary. Changes in any of these would affect the actual results secured, although the relative values would be consistent with reasonable limits of test. The general relation secured by the given conditions of test, is consistent with the results obtained by the compressive tests. It is felt that this relation, among others, offsets objection to the use of the 6 in. x 12-in. cylinder in tests of concrete made with coarse aggregate graded up to 2¼ in. especially where sufficient number of specimens are made.

Standard laboratory practice was followed throughout each investigation. All mixtures were tested for given consistency by means of the flow table. The specimens for compression tests were capped top and bottom with neat cement, using machined iron plates to secure a smooth surface, and just before testing were ground to a true plane on a revolving disk with powdered emery. The specimens were stored in a damp closet up to the 28-day period, and then removed to damp sand storage until tested. One specimen was made of each mix and age, etc., each day for ten consecutive days. Before each day's mixing began, sufficient of each material for the program for that day was mixed and stored.

All results given in compression represent the average of ten cylinders. The impact test results represent the average of from 10 to 36 specimens.

In order to avoid confusion the results from which the curves are platted are left off, except where the points do not lay practically on the curve. In the latter cases they are put on the chart for reference. In Fig. D-1, the theoretical difference between the strength of each gradation and those surrounding it, being less than the ordinary test error, led to the contours being drawn from the value secured by averaging each set of three triangularly adjacent gradations.



IMPACT TESTING MACHINE FOR CONCRETE.

It is assumed that the compressive and impact tests approximate, or are somewhat related to, the stresses occurring in a concrete pavement. While the pavement seldom fails in direct compression, or is subjected to direct blow such as delivered in the impact test, the rumbling of heavy traffic over the surface approaches either one or the other in a more or less approximate manner.

Both methods of test develop a number of important inter-relations regarding cement and aggregate, which either compensate for or add to, the effect of others. They all lead to the consideration of certain principles which, if put into effect, should produce economic concrete.

For convenience the data are classified under the following headings:

1. METHOD OF PROPORTIONING MATERIALS.
2. VARIATION IN RELATIVE STRENGTH WITH INCREASED ACTUAL STRENGTH.
   (a) With different cement content.
   (b) With different sizes of coarse aggregate.
   (c) With different ratio of fine to coarse aggregate.
3. RELATION BETWEEN IMPACT AND COMPRESSION TESTS.
4. VARIATION IN TYPES OF COARSE AGGREGATE.
5. APPLICATION.

METHODS OF PROPORTIONING MATERIALS.

Assuming field concrete in place for predetermination of quality in the laboratory, tests should be conducted to eliminate all variables encountered by error or manipulation. Concrete in place is designed to contain a given amount of cement and aggregate,—generally divided into two classes:—i. e., fine and coarse. The latter are generally specified by minimum and maximum limits. The gradation of the coarse aggregate, within the usual limits, has a considerable effect on the strength of the concrete, and also varies the amount of concrete secured when added to definite quantities of fine aggregate and cement. Such variables require close control when testing for the effect of the different ingredients on the strength of concrete.

There are four major variables in a given volume of concrete—cement, fine aggregate, coarse aggregate, and consistency. When testing to determine the effect of any one of these variables on the strength of concrete, it is necessary that the others be kept constant if proper comparison is made. It is obvious that accurate tests for two variables cannot be made at the same time. The method used in proportioning the materials for the tests included in this paper eliminated all variables except those designated for comparison.

By a consideration of these four variables, it is possible to differentiate between tests for quality and those for quantity. The use of arbitrary methods in proportioning cement and aggregate for either test or for prac-

tical purposes conflict in this respect. This is on account of such factors as the variation in the bulking effect of different gradations of fine and coarse aggregate producing a variable in the volume of concrete obtained. They are effective in changing the ratio of each material to each other material, which has an important bearing on not only the strength but the unit cost of the concrete. A proper comparison of materials for economic concrete should take into consideration not only the unit cost of the different materials, but the quantity of each necessary to produce a given volume of concrete.

The difference in quality of concrete mixtures should be determined by tests of concrete. The difference in cost should be computed.

The method used in these investigations required considerable preliminary work to determine the exact quantities of each ingredient necessary to secure given volume of concrete. The results obtained, however, more than justify the additional time and expense.

While it is desirable to consider fine and coarse aggregate under a single division, it would widen the limits, as such, beyond control. The design of concrete mixtures, should, however, consider the aggregate under a single heading. Field application should divide the aggregate, from minimum to maximum, into several sizes, preferably bearing direct mathematical relation to each other. The number of sizes selected should be consistent with the range between the minimum and maximum limits, as well as with the importance of strength and uniformity of the concrete required for the given structure.

A given volume of inert aggregate, regardless of gradation, in itself, will have no strength. On the other hand the same volume containing all cement will have highest strength. By making these assumptions in outlining tests, there are two extremes obtained which are constant in all cases. This is true regardless of the mixtures of cement and aggregates used between these extremes. With the volume of mixed concrete and the consistency constant, the difference in relative strength between any number of gradations of aggregate can be indicated by the divergence of the respective curves, platted from the values obtained by increasing the cement content from 0 to 100 per cent and decreasing the aggregate content accordingly. The latter is necessary to maintain the constant volume of mixed concrete. Fig. 2 shows typical curves by this method.

All cement is proportioned (dry) in volumetric ratio to the finished concrete. The percentages, by this method, applied to arbitrary field mixtures will approximate the following: 1 : 3 : 6 = 15 per cent; 1 : 2½ : 5 = 18.5 per cent; 1 : 2 : 4 = 22 per cent; 1 : 2 : 3 = 26 per cent; 1 : 1½ : 3 = 28 per cent.

The tests include practically every gradation of coarse aggregate used in normal concrete work. They also furnish data on the effect of variation in the cement content as well as in the ratio of fine to coarse aggregate. From the results secured, and by reasonable interpretation of the curves,

etc., as indicated by definite tendencies, which in many cases are of as much importance as the curves themselves, the following conclusions are offered:

(1) Concrete should be proportioned.

(a) With a definite amount of cement per volume of completed mixture.

(b) By combining definite amounts of several definite sizes of aggregate.

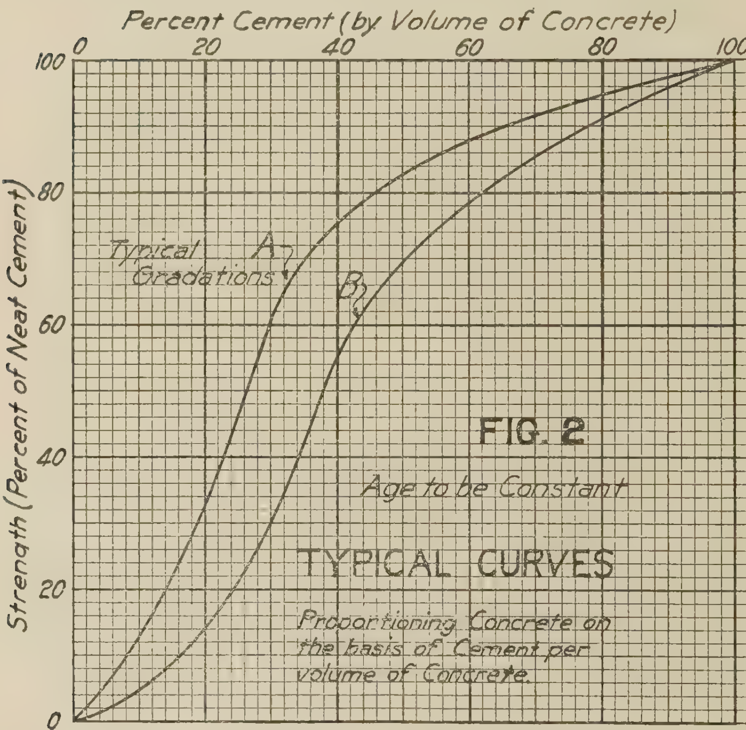


FIG. 2.—TYPICAL CURVES SHOWING CEMENT CONTENT FROM 0% TO 100%.

Laboratory work by this method will generally require section of curves including cement contents from 15% to 40% in five per cent units depending on the type of investigation.

(c) With reference to a desired strength at a given age.

(d) By regulation of either the amount of cement, the size and type of the aggregate, the ratio of fine to coarse aggregate, or combinations of these, to secure the desired quality under "c" economically.

(e) All tests for comparison or determination of quality should be made from concretes of equal volumes and consistencies.

(2) *With all other factors equal*

(a) The compressive strength and the resistance to impact of concrete are increased as the size of the coarse aggregate is decreased. The minimum limits appear to be governed by the amount and quality of cement in the mix.

(b) The relative increase in compressive strength of the over-sanded mixtures, while higher at the early period (28 days), is lower at later ages (or other normal factors causing increased actual strength).

(c) This effect (b) is greater as the size of the coarse aggregate is decreased.

(d) The relative compressive strength of concretes containing the smaller sizes of coarse aggregate increases at a greater rate than those containing the larger sizes.

(e) The relative compressive strength of concretes having lower cement contents increases at a greater rate than those having a higher cement content.

(3) *There is an economic relation between the amount of cement in the mixture and the maximum size of the coarse aggregate used.*

(4) *Age or other causes whereby the strength of the concrete is normally increased, affects changes in the relative strength of concrete mixtures.*

(5) *Tests at the early ages (7 and 28 days) are suited only to determine the quality of concrete at such ages. They do not truly indicate the proper relation between different mixtures at the later ages. In the leaner mixtures they have a tendency to be more misleading than in the richer.*

(6) *In normal concrete mixtures there appears to be a definite relation between the resistance to impact and the compressive strength.*

(7) *The quality of the coarse aggregate affects both the compressive strength and the resistance to impact of concrete.*

(8) *With different types of coarse aggregates, size may be adjusted to compensate for difference in quality of concretes having equal cement contents. Cement may also be used as a means to accomlish the same purpose.*

(9) *The effect of change in relative strength on the actual strength of concrete, containing various amounts of cement, gradations, and types of aggregates, etc., should be considered especially in the design of concrete mixtures for highway construction.*

(10) *By making tests of various types of aggregates in concretes rich in cement, it is possible to approximate the relative values they will possess in the more normal mixtures at the later periods. The smaller size of aggregate (3/8 in.-3/4 in.) should be used for this work.*

OVER-SANDED MIXTURES.

The tests indicate that as the ratio of fine to coarse aggregate is increased in normal concrete mixtures, a higher early strength is obtained for a given cement content. The rate of increase with age is lower than in the mixtures having a lower ratio of fine to coarse aggregate.

In Fig. C-1 the actual strengths of the various concrete mixtures, at the ages of 28 days and one year, are platted against cement content. At the age of 28 days the over-sanded mixtures show higher actual strength than those having a fine-coarse aggregate ratio of 1: 2, but the curves coincide at the one-year period. Attention is called to Fig. C-3 in which the average relative increase in strength of all gradations of aggregate from Series 23-106 containing 25 per cent of cement, are platted in connection with Series 23-100. The ratio of fine to coarse aggregate in Series 23-100 is slightly higher than that in Series 23-106. It will be noted that although the data refer to two separate investigations, the same tendency exists in favor of the lower fine-coarse aggregate ratio as both concretes increase in actual strength. Fig. C-4, Series 23-46, gives another indication of the effect of change in relative strength on the actual strength of the over-sanded mixtures. In this case the strengths were obtained by increasing the cement content instead of by age. The age of test (28 days) is such that minimum changes would be expected. This figure also shows the effect of the gradation of coarse aggregate on the rate of change in both actual and relative strengths. It will be noted that the lower fine-coarse aggregate ratio tends to produce the stronger concrete as the actual strength of all mixtures is increased either by age or higher cement content. They indicate that for highest ultimate strength, the lower ratios of fine to coarse aggregate are more desirable. Also that the smaller gradations are more desirable for highest strength.

RESISTANCE TO IMPACT.

Figs. E-1 and E-2 contain the results of impact tests on concrete. The data are taken from Series 23-56 and 23-110. In Fig. E-1 are platted the results of impact tests of concrete containing two types of aggregates, each of three different sizes. It will be noted that the same general relation is found between the different sizes of coarse aggregate as is determined in the compression tests. This relation, as indicated by given test conditions, is platted in Fig. F-1. The compression tests, containing 30 per cent of cement are taken from Series 23-100 and those containing 25 per cent of cement are taken from Series 23-106. The effect of cement on the resistance to impact of concrete is given in Fig. E-2 (Series 23-56). In Fig. F-2 is the relation between the compressive strength and resistance to impact found by Series 23-110.

COMPRESSIVE TEST RESULTS.

In Figs. D-1, D-2, D-3, D-4, D-5 and D-6 are platted the actual and relative compressive strengths secured by variation in the cement content, size, and gradation of coarse aggregate, as indicated by tests at different ages up to one year. The data are obtained from Series 23-100, 23-106, and 23-56.

Fig. D-1 contains the relative strengths of concrete made from 21 gradations of coarse aggregate containing a given cement content and tested at the one-year period. The gradations were selected with definite relation to each other on a trilinear chart basis. The tests at earlier ages in this investigation indicated but slightly the effect of gradation of coarse aggregate upon the ultimate strength of the concrete. The changes in relative strength are so great that the early tests may be misleading in respect to the ultimate strength as indicated by tests at the one-year period. An example of this is given in Fig. D-2. In this figure four assorted gradations from large to small, from the 21 gradations originally used in series 23-106, are selected to bring out the predominance of the smaller size of coarse aggregate on the basis of ultimate compressive strength. This figure is with reference to the change in relative strength of concrete having a given cement content (25 per cent).

Changes in relative strength with variation of the cement content and the coarse aggregate constant are given in Fig. D-3. (Series 23-110). While these data contain concretes of two ratios of fine to coarse aggregate, which in themselves affect the difference in relative strength, the points on the curve, when platted with reference to the respective cement contents, fall sufficiently in line with each other to allow deductions to be made. Fig. C-3 also shows this relation when the average of all gradations of coarse aggregates is taken for the basis of platting (Series 23-100). The extremes encountered under the given conditions of test are platted in Fig. D-4. The relation of the several sizes of coarse aggregates and different cement contents, with reference to the effect of relative strength on the actual strength, is given in Fig. D-5. The definite tendency to bring out the importance of the smaller sizes for highest strength is again brought out in Fig. D-6 (Series 25-56) in which, as in Fig. C-4, the tests were made at the early age and additional cement was used to develop the actual strength of the concrete.

VARIATION IN TYPES OF COARSE AGGREGATE.

In Fig. D-8 the average values of tests of different types of coarse aggregate are charted. They consist of compression and toughness (Page Impact) tests of the coarse aggregate and the same of concrete containing 25 per cent (by volume) cement, in which the different types of coarse aggregate were used. The results should not be interpreted literally regarding type, as the variation of individual aggregates in each type is sufficient to cause a range considerably greater than the margin between the different types of aggregate. They are intended only to bring out the

fact that such difference appears to exist between types of coarse aggregate and that this difference is not detected at early ages in normal concrete mixtures by the compression test.

For convenience, alone, the groups are divided as follows:

1. Copper smelter slag
2. Relatively soft marl
3. Partly decomposed granite containing excess of biotite
4. Average of all gravels used
5. Average of all blast-furnace slags
6. Average of all limestones
7. One average granite
8. Average of trap rock and extremely hard quartzite
9. Cement clinker

It may be noted that the average of all toughness (Page Impact) tests of the aggregate are fairly consistent with the average results secured by the impact tests of concrete containing them.

In order to determine the actual value of different types of aggregate in concrete it would be necessary to make the longer time tests of each particular kind of aggregate in the proportions of mixture proposed for use. There is the possibility of an accelerated test being developed, in compression, by the use of greater cement content and smaller size of coarse aggregate to furnish the desired indications. The impact test at the earlier ages indicates a greater difference in coarse aggregates than the compression tests.

Although crushed granite was used in most of the investigational work, it was compared with gravel in a few of the investigations. These tests were made at the twenty-eight day period only. Fig. D-7 is typical of the results secured.

APPLICATION.

To a certain extent, the foregoing work is typical in character. It illustrates the action of cement and coarse aggregate only. Several thousand specimens of mortars, including tension, compression, and transverse tests, are available for analysis but the results would be too voluminous for inclusion in this paper.

The conclusions from this paper appear to explain reasons for the variation encountered in field concrete, such as are caused by inaccurate proportioning of the aggregates and cement. The results secured, with reference to the effect of the larger sizes of coarse aggregate, are consistent with test results of cores taken from several hundred miles of concrete pavement. The effect of type of aggregate is many times apparent from both observation of such construction where different types are accepted and used under the same specification, as well as by careful records of such by means of crack surveys, etc.

It is not the intent to recommend that the smallest size of coarse aggregate be used arbitrarily. To do this, in order to develop the highest

strength, will necessarily increase the cost of not only the aggregate, but also that of the cement, as more will be necessary to obtain a balanced mixture. It also does not mean that the cement content should be kept at the minimum. An increase in the cement content of normal mixtures will many times allow a constant quality of concrete to be obtained with less expensive aggregate; and also in proportion to its cost may be an economic way of increasing quality. The principle is to balance the mixture in order that the highest quality in concrete will be secured economically with a given amount of cement and aggregate.

By the use of a suitable method for proportioning the aggregate scientifically, and obtain an economic balance between the size and gradation of the aggregate, and the cement content, a decrease in actual cost may be accompanied by an increase in quality and uniformity of the finished concrete.

Such a method, designed to produce a mixture containing a definite quantity of cement and each size of aggregate to make a given volume of concrete, of a given consistency, will eliminate such variations as usually occur. It will eliminate difficulties encountered by present methods from the bulking of fine aggregate due to moisture, segregation of coarse aggregate in stockpiles, etc., or other causes for variation in quality and cost, all of which have considerable effect on economic concrete.

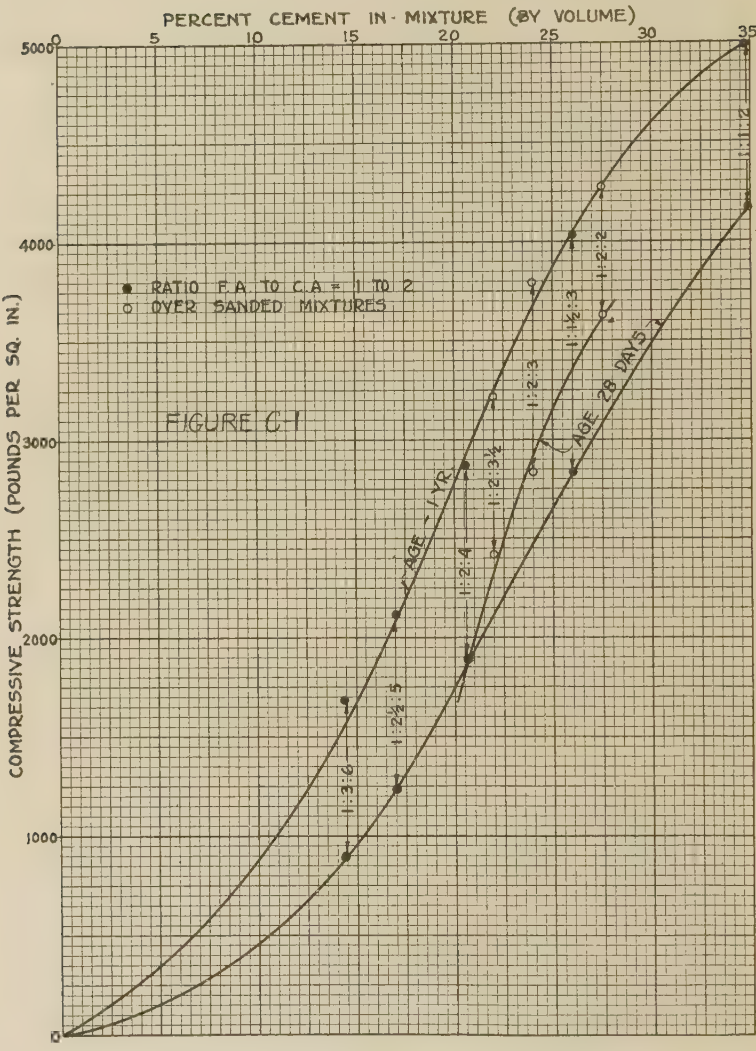


FIG. C-1.—CURVES SHOWING THE HIGHER ACTUAL STRENGTHS OF THE OVER-SANDED MIXTURES AT THE EARLIER AGE AND THE RELATIVE LOSS IN STRENGTH AT THE LATER AGE.

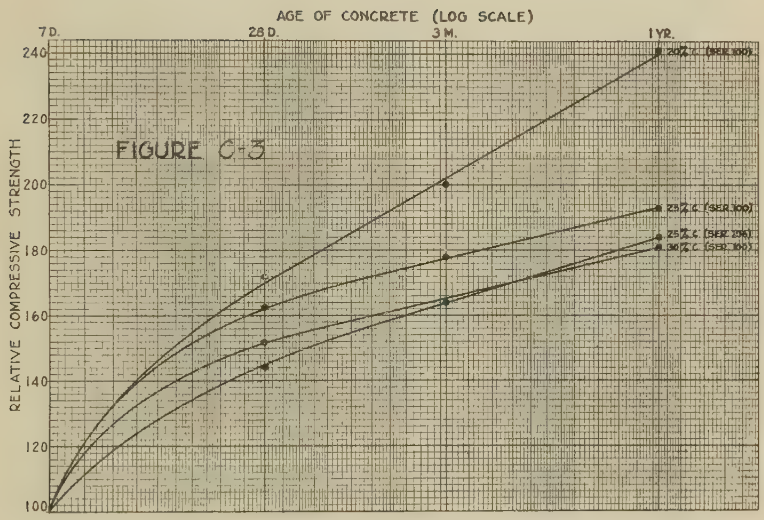


FIG. C-3.—CURVES SHOWING THE AVERAGE EFFECT OF CEMENT ON THE RELATIVE STRENGTH OF CONCRETE. EACH CURVE IS THE AVERAGE OF GIVEN CEMENT CONTENT AND ALL GRADATIONS OF COARSE AGGREGATE.

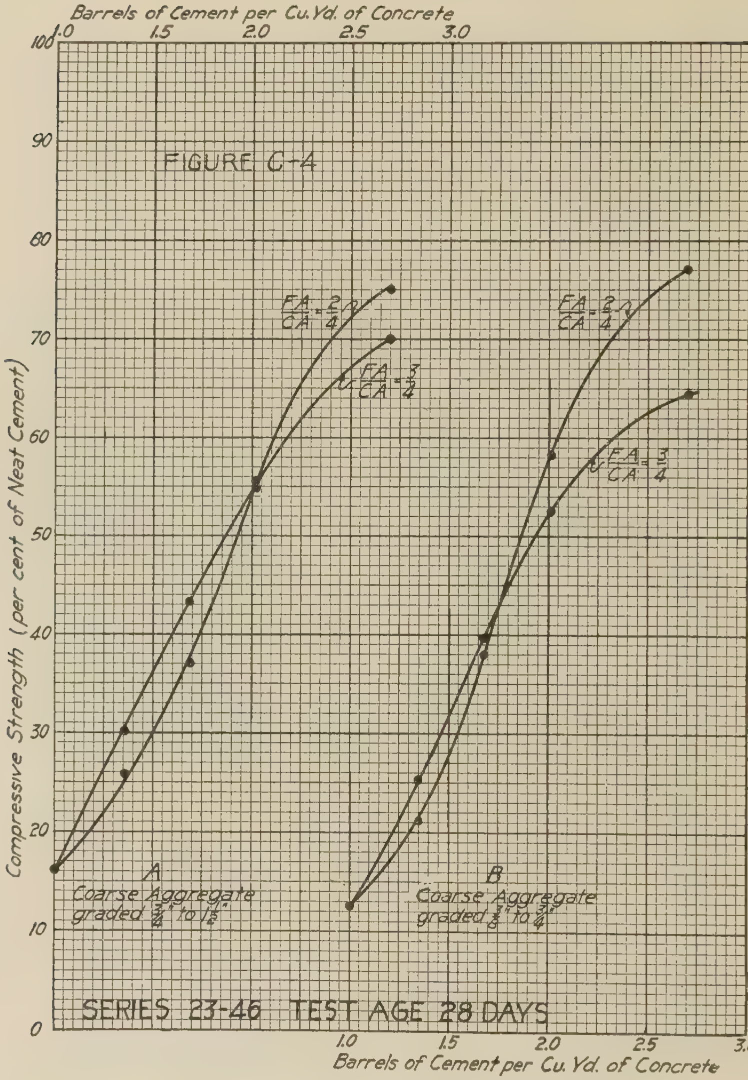


FIG. C-4.—CURVES SHOWING THE EFFECT OF DIFFERENT CEMENT CONTENTS ON THE COMPRESSIVE STRENGTH OF CONCRETES CONTAINING DIFFERENT SIZES OF COARSE AGGREGATES AND DIFFERENT RATIOS OF FINE TO COARSE AGGREGATE.

Note the greater effect in the smaller size of coarse aggregate.

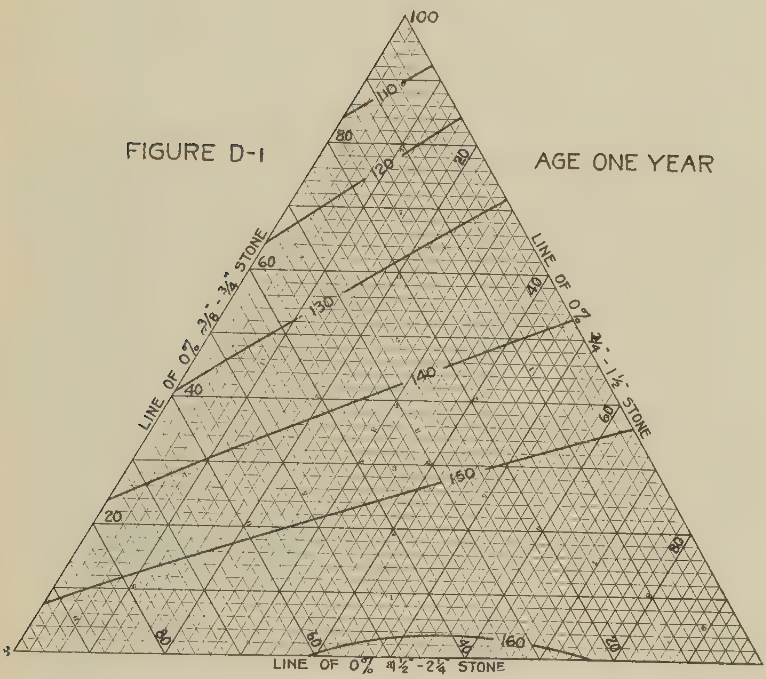


FIG. D-1.—TRILINEAR CHART SHOWING RELATIVE COMPRESSIVE STRENGTH OF CONCRETE CONTAINING DIFFERENT GRADATIONS OF COARSE AGGREGATE AT THE AGE OF ONE YEAR.

Cement content 25%, and ratio of fine to coarse aggregate 1 : 2. Note the general increase in strength as the size of the coarse aggregate decreases.

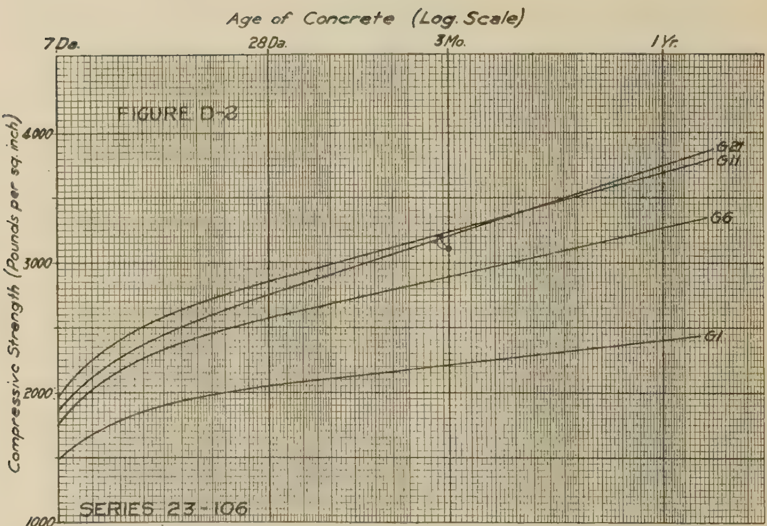


FIG. D-2.—CURVES SHOWING THE RELATIVE COMPRESSIVE STRENGTH OF CONCRETE CONTAINING GIVEN CEMENT CONTENT AND FOUR ASSORTED GRADATIONS OF COARSE AGGREGATE.

Showing the greater relative increase in strength of the smaller sizes as the concrete increases in actual strength.

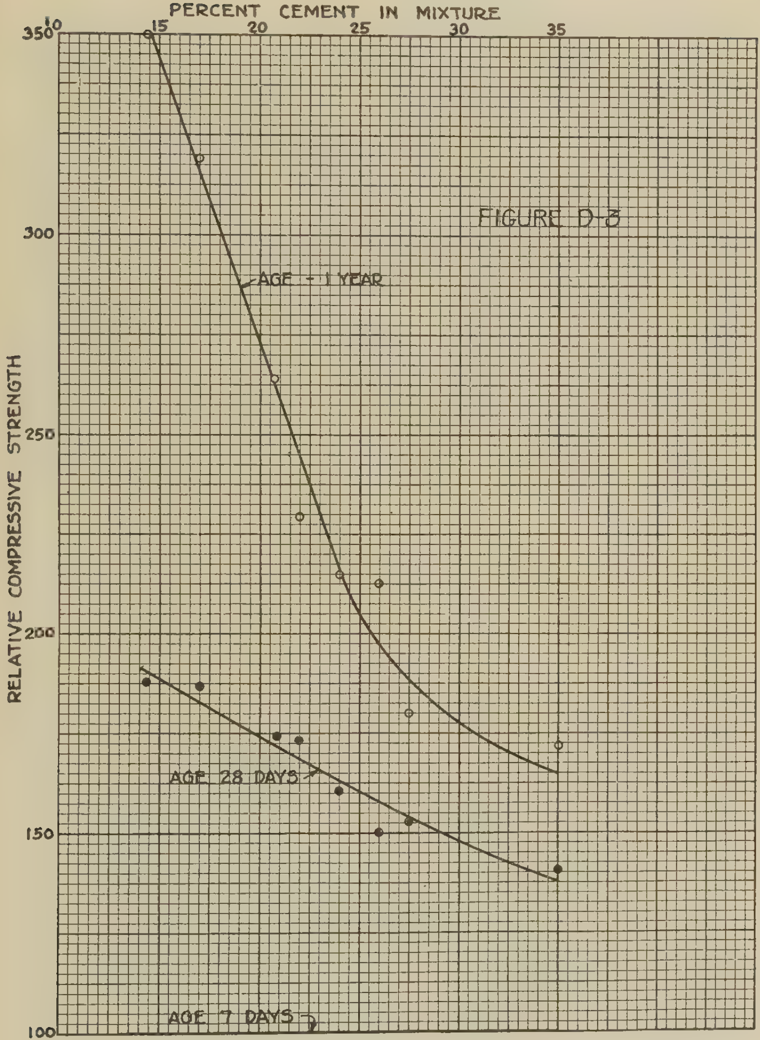


FIG. D-3.—CURVES SHOWING THE EFFECT OF CEMENT ON THE RELATIVE INCREASE IN COMPRESSIVE STRENGTH WITH AGE, OF CONCRETE CONTAINING GIVEN SIZE OF COARSE AGGREGATE.

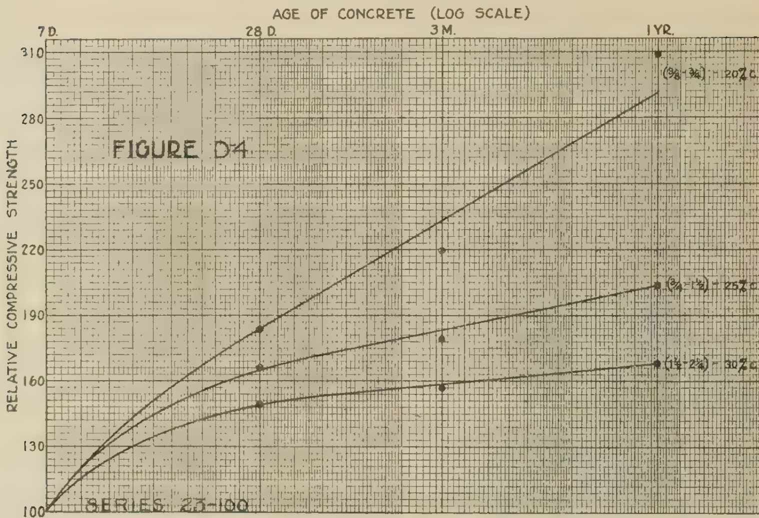


FIG. D-4.—CURVES SHOWING THE EXTREMES ENCOUNTERED IN RELATIVE STRENGTHS OF CONCRETE MIXTURES WITH COMBINED ACTION OF CEMENT CONTENT AND COARSE AGGREGATE GRADATION.

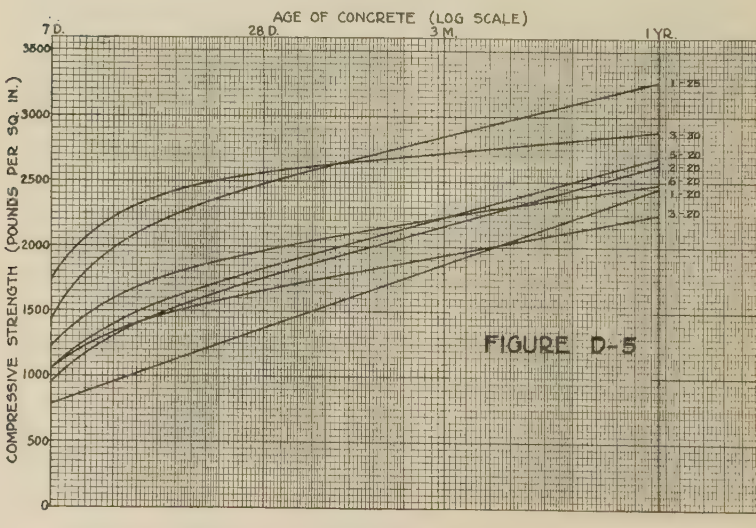


FIG. D-5.—CURVES SHOWING THE EFFECT OF CHANGES IN RELATIVE STRENGTH ON THE ACTUAL STRENGTH OF CONCRETE MIXTURES OF VARIOUS AGGREGATE GRADATIONS AND CEMENT CONTENTS (SERIES 23-100).

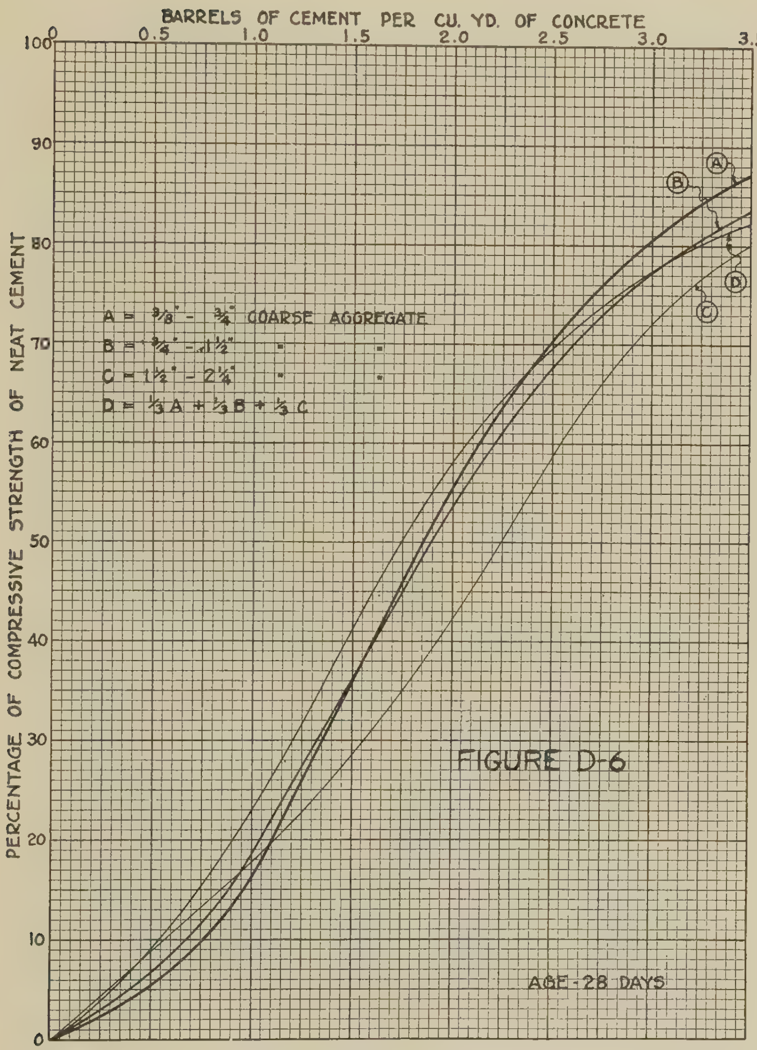


FIG. D-6.—CURVES SHOWING THE CHANGES IN THE ACTUAL COMPRESSIVE STRENGTH OF CONCRETE CONTAINING VARIOUS SIZES OF COARSE AGGREGATE WITH VARIATION IN CEMENT CONTENT.

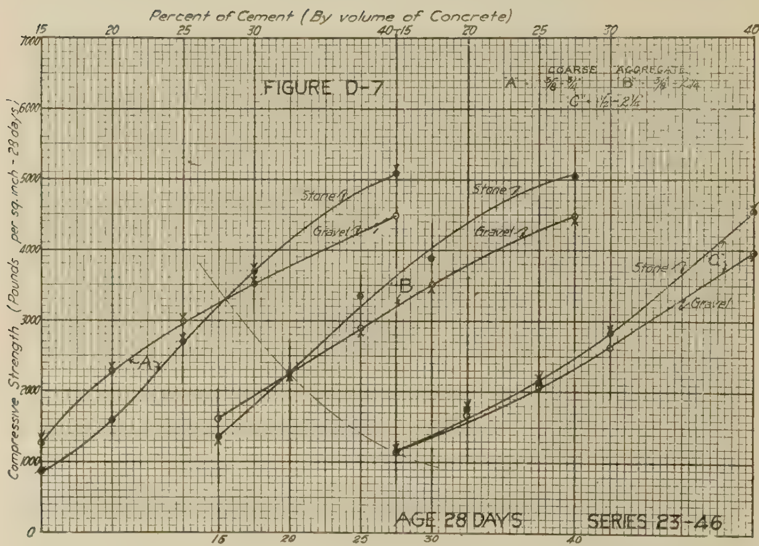


FIG. D-7.—CURVES SHOWING THE EFFECT OF VARIATION IN CEMENT CONTENT ON THE ACTUAL STRENGTH OF CONCRETE MIXTURES CONTAINING TWO TYPES (CRUSHED GRANITE AND GRAVEL) AND THREE SIZES OF COARSE AGGREGATE.

Note the points cross at lower points as the size of the coarse aggregate increases.

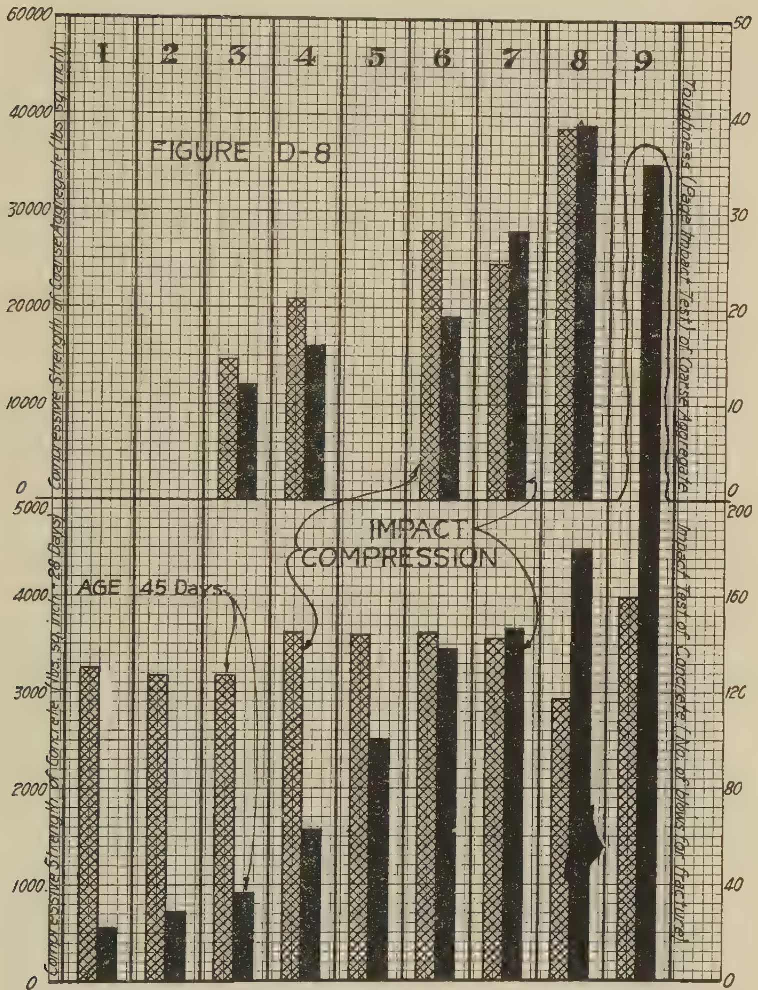


FIG. D-8.—CHART SHOWING THE AVERAGE RESULTS SECURED BY DIFFERENT TYPES OF COARSE AGGREGATE BY IMPACT AND COMPRESSION TESTS, BOTH SEPARATELY AND IN CONCRETE.

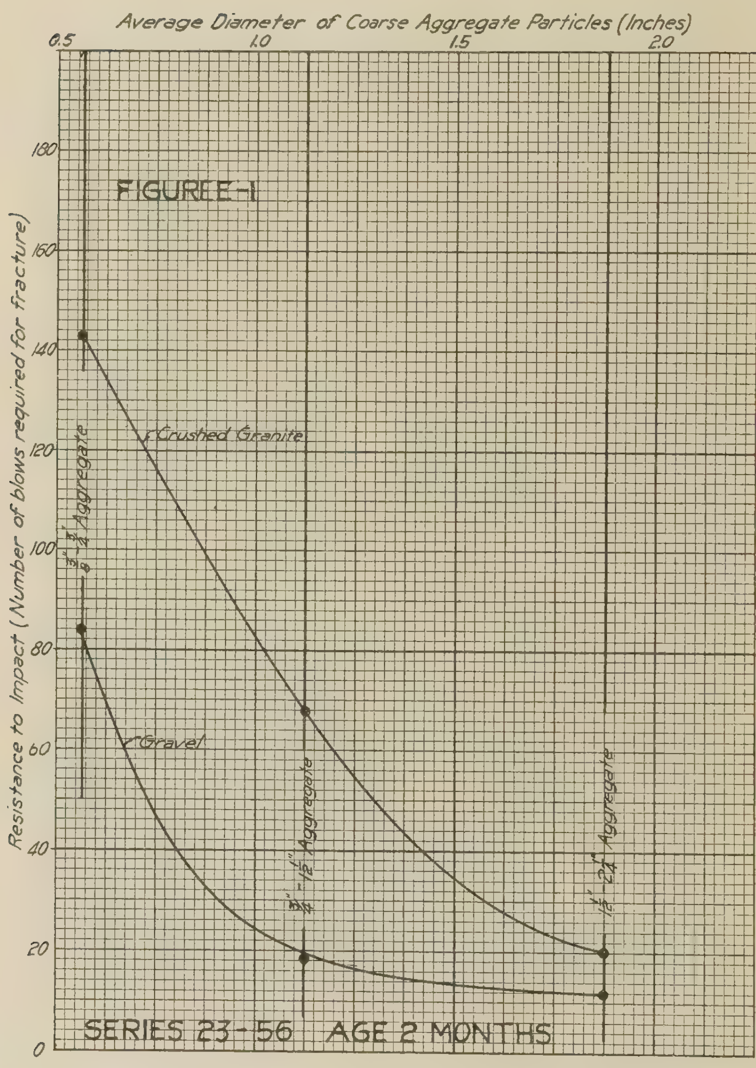


FIG. E-1.—CURVES SHOWING THE EFFECT OF IMPACT ON CONCRETE MIXTURES CONTAINING TWO TYPES OF COARSE AGGREGATE (CRUSHED GRANITE AND GRAVEL) OF THREE DIFFERENT SIZES.

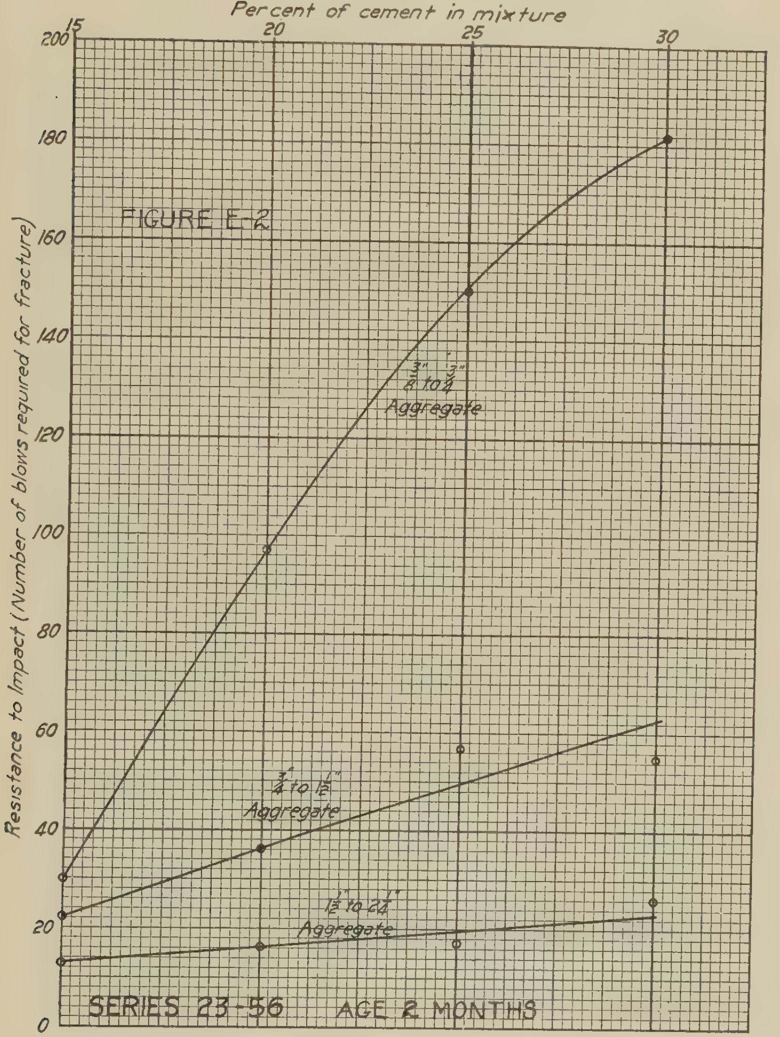


FIG. E-2.—CURVES SHOWING THE EFFECT OF CEMENT ON THE RESISTANCE TO IMPACT OF CONCRETE CONTAINING DIFFERENT GRADATIONS OF COARSE AGGREGATE.

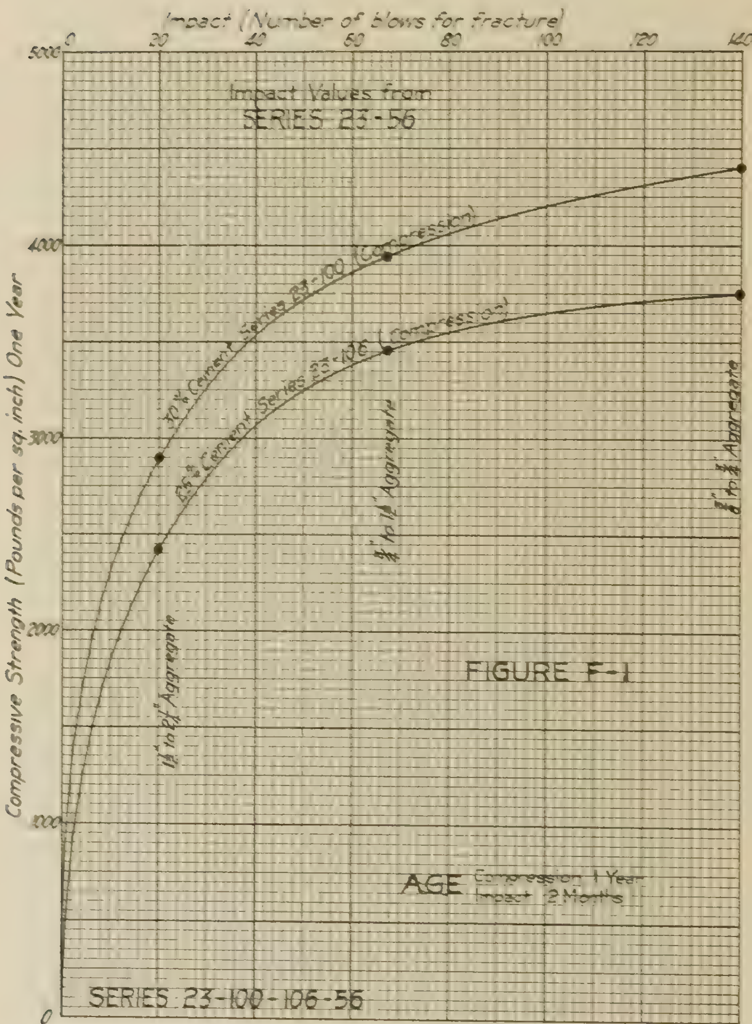


FIG. F-1.—CURVES SHOWING TYPICAL RELATION BETWEEN IMPACT AND COMPRESSIVE TESTS OF CONCRETE MIXTURES.

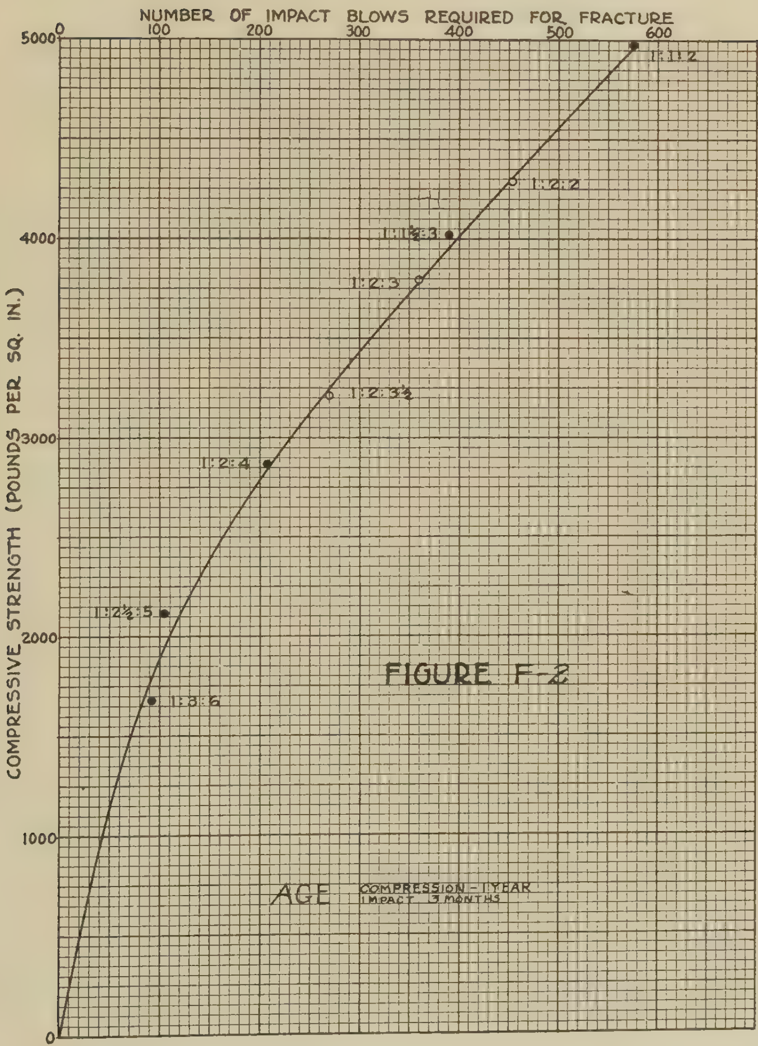


FIG. F-2.—CURVE SHOWING THE RELATION BETWEEN COMPRESSIVE STRENGTH AND THE RESISTANCE TO IMPACT OF VARIOUS ARBITRARILY PROPORTIONED CONCRETE MIXTURES.

TABLE OF GRADATIONS OF COARSE AGGREGATE—SERIES 23-100

Ratio Fine to Coarse Aggregate = 2:3½ (by volume)

GRADATION NO.	⅜–¾ in.	¾–1½ in.	1½–2¼ in.	Total
1	100	..	..	100
2	..	100	..	100
3	..	..	100	100
4	33⅓	33⅓	33⅓	100
5	50	50	..	100
6	..	50	50	100
7	50	..	50	100

COMPRESSIVE TESTS OF CONCRETE—SERIES 23-100

Gradation Number	Percent Cement	7 DAYS Lb. sq. in.	7 DAYS Rel.	28 DAYS Lb. sq. in.	28 DAYS Rel.	3 MOS. Lb. sq. in.	3 MOS. Rel.	1 YEAR Lb. sq. in.	1 YEAR Rel.
3	20	1057	100	1721	163	1866	177	2269	215
1	20	798	100	1469	184	1751	219	2462	309
6	20	1228	100	1965	160	2247	183	2476	202
4	20	1073	100	1915	178	6163	202	2584	241
2	20	987	100	1775	180	2118	215	2640	267
5	20	1050	100	1842	175	2221	212	2686	256
7	20	1317	100	2190	166	2513	191	2719	206
3	25	1593	100	2472	155	2515	158	2775	174
3	30	1717	100	2598	151	2696	157	2901	169
1	25	1445	100	2479	172	2855	198	3257	225
6	25	1827	100	2902	159	3076	168	3293	180
4	25	1833	100	2907	159	3341	182	3348	183
7	25	1823	100	2882	158	3215	176	3401	187
2	25	1695	100	2823	167	3030	179	3466	204
5	25	1833	100	3129	171	3444	188	3678	201
2	30	2158	100	3341	155	3511	163	3944	183
7	30	2314	100	3452	149	3788	164	4107	177
5	30	2445	100	3619	148	3940	160	4299	176
6	30	2306	100	3534	153	3663	159	4312	187
4	30	2445	100	3751	153	4159	170	4321	177
1	30	2255	100	3467	154	3940	175	4400	195

TABLE OF GRADATIONS OF COARSE AGGREGATE—SERIES 23–106

Ratio Fine to Coarse Aggregate = 1:2 (by volume) 25 per cent Cement (by Dry Volume)

GRADATION NO.	⅜–¾ in.	¾–1½ in.	1½–2¼ in.	Total
1	00	00	100	100
2	00	20	80	100
3	20	00	80	100
4	00	40	60	100
5	20	20	60	100
6	40	00	60	100
7	00	60	40	100
8	20	40	40	100
9	40	20	40	100
10	60	00	40	100
11	00	80	20	100
12	20	60	20	100
13	40	40	20	100
14	60	20	20	100
15	80	00	20	100
16	00	100	00	100
17	20	80	00	100
18	40	60	00	100
19	60	40	00	100
20	80	20	00	100
21	100	00	00	100

COMPRESSIVE TESTS OF CONCRETE—SERIES 23–106

Gradation Number	7 Days Lb. sq. in.	7 Days Rel.	28 Days Lb. sq. in.	28 Days Rel.	3 Mos. Lb. sq. in.	3 Mos. Rel.	1 Year Lb. sq. in.	1 Year Rel.
1......	1470	100	2058	140	2153	146	2416	164
2......	1623	100	2242	138	2540	157	2753	170
4......	1610	100	2201	137	2611	162	2940	183
3......	1808	100	2467	136	2568	142	3017	167
7......	1918	100	2750	143	3009	157	3076	160
6......	1732	100	2583	149	2859	165	3291	190
5......	1789	100	2630	147	2871	160	3314	185
8......	1764	100	2462	140	3032	172	3318	188
20......	1776	100	2678	151	3197	180	3357	189
14......	1832	100	2842	155	3216	176	3453	188
16......	1827	100	2520	138	3104	170	3470	190
12......	1788	100	2746	154	3192	179	3515	197
13......	2026	100	2922	144	3344	165	3684	182
9......	2118	100	3031	143	3340	158	3687	174
11......	1967	100	2851	145	3102	158	3704	188
15......	1978	100	2839	144	3326	168	3735	189
21......	1858	100	2759	148	3099	167	3748	202
10......	1994	100	2887	145	3304	166	3766	189
17......	1962	100	2864	146	3237	165	3798	194
18......	2050	100	2903	142	3356	164	3840	187
19......	2043	100	2888	141	3244	159	3914	192

COMPRESSIVE AND IMPACT TESTS OF CONCRETE—SERIES 23–110

Compression

Mix	7 Days No. sq. in.	7 Days Rel.	28 Days No. sq. in.	28 Days Rel.	3 Mos. No. sq. in.	3 Mos. Rel.	1 Year No. sq. in.	1 Year Rel.	Impact 3 Mon. (No. blows for fracture)
1:3:6...	478	100	897	188	1111	232	1682	352	93
1:2½:5.	661	100	1238	187	1487	225	2108	319	106
1:2:4...	1085	100	1886	174	2223	205	2866	264	209
1:2:3½.	1401	100	2433	173	2657	190	3214	229	271
1:2:3...	1766	100	2839	161	3049	173	3798	215	361
1:1½:3.	1895	100	2838	150	3330	176	4028	213	391
1:2:2...	2377	100	3634	153	3902	164	4271	180	456
1:1:2...	2977	100	4183	141	4312	145	5120	172	583

DISCUSSION.

Mr. Walker.

Stanton Walker.—Mr. Hutchinson's paper may seem to contradict the general conclusions drawn from thousands of tests carried out at the Structural Materials Research Laboratory, with which I am associated, and at other laboratories. His tests indicate that coarse aggregates of relatively fine gradings produce higher strengths than those of coarse gradings, while our tests and many others, show the reverse of this to be generally true.

However, from a close examination of the gradings of aggregate used by Mr. Hutchinson, it seems apparent that many of them, particularly the coarse ones, contained too little sand to produce a workable mix. These under-sanded mixtures probably gave relatively low strengths which would account for the failure of Mr. Hutchinson's tests to check the results of other tests. If the aggregates in Series 23-106 are considered on the basis of fineness modulus it is found that about one-third of them give fineness moduli higher than would be expected to produce suitable workability for concrete containing 25 per cent of cement. Our tests show a rapid falling off in strength when aggregates too coarse for proper workability are used. Even those aggregates giving values lower than the upper limit of fineness modulus which should be permitted, were so coarse that, in our opinion, erratic results should be expected.

Only the grading of the coarse aggregate is given in Mr. Hutchinson's paper. The fineness moduli of the mixed aggregates were calculated on the assumption that an average sand of fineness modulus about 3.00 was used.

The fineness modulus is a measure of the grading of aggregate calculated from the sieve analyses. The coarser the aggregate, the higher the fineness modulus. For a complete description of this function, see "Design of Concrete Mixtures," Bulletin 1 of the Structural Materials Research Laboratory, Lewis Institute, Chicago. The maximum permissible values of fineness modulus for different mixtures and gradings of aggregate are given in Table 3 of this Bulletin. Aggregates having fineness moduli higher than these values are too coarse for proper workability.

Prof. Richart.

F. E. Richart.—Mr. Hutchinson's observations regarding the effect of the size of the coarse aggregate are substantiated to some degree by some tests conducted recently at the University of Illinois. A series of tests of the compressive strength of concrete mixtures taken from the table of proportions given in the report of the Joint Committee on Specifications for Concrete and Reinforced Concrete includes concretes made with three gradations of sand and three of coarse aggregate, three consistencies and three designed strengths. The coarse aggregates used were gravels of gradations designated as No. 4 to ¾-in., No. 4 to 2-in., and ¾ to 2-in. The principal feature of the tests was that the concrete made with the

first coarse aggregate gave unexpectedly high strengths, and that containing the last, the ¾ to 2-in. material, gave relatively low strengths. That is, this rather uniformly graded coarse aggregate, in which the particles were of large size and which had a high fineness modulus, gave low strengths, while of the better graded aggregates, the one having the smaller maximum size gave the best results. These unexpected variations in strength are not only variations from the designed strength, but also variations to a marked degree from the curves expressing the relation between strength and water-cement ratio and voids-cement ratio.

It would seem that the explanation might lie in the extent and condition of the surface of the coarse aggregates. While all of the aggregates were from the same source, the surface moduli, in the order mentioned above were 1.27, 0.92, and 0.35, so that the coarse aggregate giving the least strength had only about one-fourth as much surface area as the one giving the highest strength. The strength of the concrete may also be affected by the extent of the bearing contact and interlocking between particles of coarse aggregate, in a way similar to that observed in the action of railway track ballast.

W. A. Slater.

W. A. SLATER.—I am constrained to repeat a part of my discussion of day before yesterday on Mr. Watson's paper. We have made tests of concrete which were similar, I think, to those which Prof. Richart refers to. The size of the coarse aggregate varied. We had No. 4 to ¾ in., No. 4 to 2 in., and ⅜ in. to 1½ in., and we found the same results that Prof. Richart refers to, that the smaller coarse aggregate gave concrete with a higher strength than did the larger coarse aggregate, almost without exception. We were puzzled until we examined them by the device I previously mentioned of throwing out all the mixes in which the fine aggregate was less than half the coarse aggregate, and we found the rest lined up very well. The mixes which were thrown out were generally those using the larger coarse aggregate. In other words, I might say that we out-Walkered Walker; we got a little further down on the maximum fineness modulus than he proposes to go. If maximum fineness modulus were limited to a lower value than he has recommended the mixes using the larger coarse aggregates with the small sand content would not be permitted. Probably it was not the coarseness of the aggregate so much as the smallness of the sand content which caused the low strengths.

Prof. Richart.

PROF. RICHART.—I believe that some of the mixtures in which a very small proportion of sand was used did result in concrete of relatively low strength, however, I do not recall that these mixtures appeared particularly harsh or unworkable. Furthermore, the proportion of the ¾ to 2-in. coarse aggregate used was very moderate, being roughly 0.65 to 0.80 cubic yard, measured loose, per cubic yard of concrete; an amount much less than that used with the No. 4 to 2-in. aggregate and somewhat more than that used with the No. 4 to ¾-in. aggregate.

Mr. Hutchinson.

MR. HUTCHINSON.—Time will not permit a lengthy discussion of Mr. Walker's calculations but I believe it would be desirable to correct any misunderstanding which might occur with reference to the statement regarding the workability of the mixtures used. The leanest mixture in this work is equivalent to a normal laboratory 1 : 2 : 4 concrete and they ranged from this up to those richer than the normal 1 : 1½ : 3 and included the so-called over-sanded mixtures. They were no less workable, therefore, than any mixtures being used in concrete paving and much more workable than those generally used for concrete foundation.

We find from analyzing the results of cores drilled from up to 800 miles of finished pavement and foundation construction, and tested at ages from three months to two years, that there is the same evidence of the coarse aggregate being the limiting factor in the maximum strength, as is indicated by the data given here. Segregated aggregate seems to be detected by the variation in field concrete which averages up to 100 per cent of the minimum strength. We know from observation that the coarse aggregate is segregated and when the smaller sizes compose that in the drilled specimens, the results of the investigations given here indicate it to be the reason for the higher strengths obtained. The higher limit for strength of the concrete in these cases appears to be dependent upon the strength of the cement itself.

Coefficient of Expansion Tests on Gunite.

By M. O. Fuller.*

The discussion of the coefficient of expansion of gunite, as compared with that of concrete, resulted in some testing being performed at the Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa. As far as is known, these were the first tests ever conducted to determine the coefficient of expansion on gunite.

The specimens used in making the determinations were rectangular pieces 8 in. x 12 in. x 1¼ in., which had been cut from a gunite slab. The specimens were prepared for testing by drilling small holes near each end of the center line of the 8 in. x 2 in. face and into these holes were set very fine needle points cemented in place by a special mixture composed of one part silica cement and three parts donken clay. A specially-constructed electric furnace of the multiple-unit type was used in these tests. The following description of this furnace, and the photograph, will enable the reader to understand the operation of the furnace.

The outside dimensions of the body of the furnace measure 18 in. long, 10 in. wide and 6 in. high. It is supported by legs 3 in. high at the four corners. The cover or top, marked *F*, in the photograph, was made of a porous heat insulating material about 2 in. thick. Through the cover are two openings 8 in. long—one for the admission of light and the other for sighting on the specimen with a telescope while heating without removing the entire top. The photograph shows clearly the top and the small porcelain covers for the openings which are removed whenever observations are to be made. The walls are of the same porous material as the top, 2 in. thick and entirely covered with sheet iron. The bottom is covered with a slab of compressed asbestos. The interior space in which the specimens were placed for heating is 4 in. x 3 in. x 12 in., and the walls and bottom are made of unglazed porcelain. The furnace is of the multiple-unit resistance type, the coils being located in the bottom and walls of the furnace. The heating units are replaceable and this particular furnace operates at 250 volts, 6.6 amperes, and a maximum temperature of 1,850 deg. F. The current is introduced to the furnace by means of a plug connection in the bottom near the front. Attached to one of the side walls of the furnace are a pair of brackets supporting two vertical posts marked *D*, which in turn carry two horizontal screws for moving the telescope in a horizontal direction by means of thumb screws at *A*. The telescopes can be raised or lowered by means of screws marked *C*.

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* Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

The device for taking actual measurement of the expansion is attached to the telescope just below the eyepiece and consists of one stationary vertical cross-hair in the center of the field, a pair of movable vertical cross-hairs and a fixed horizontal cross-hair. The pair of vertical cross-hairs are moved over notched scales by means of thumb screws at *B*. These thumb screws at *B* have micrometer scales, each in turn of which corresponds to one of the small divisions on the notched scale. The value of each of these small divisions was determined to be equal to 0.01 in. The circular micrometer scale is divided into 100 equal parts, thus allowing an actual reading of 0.0001 in. and an estimated reading of one-tenth of that or 0.00001 in. After the stationary vertical cross-hairs have been adjusted to the needle points of the specimen by means of the screws at *A*, they remain in that position throughout the test. The elongation is measured entirely by the notched scale and micrometer screw.

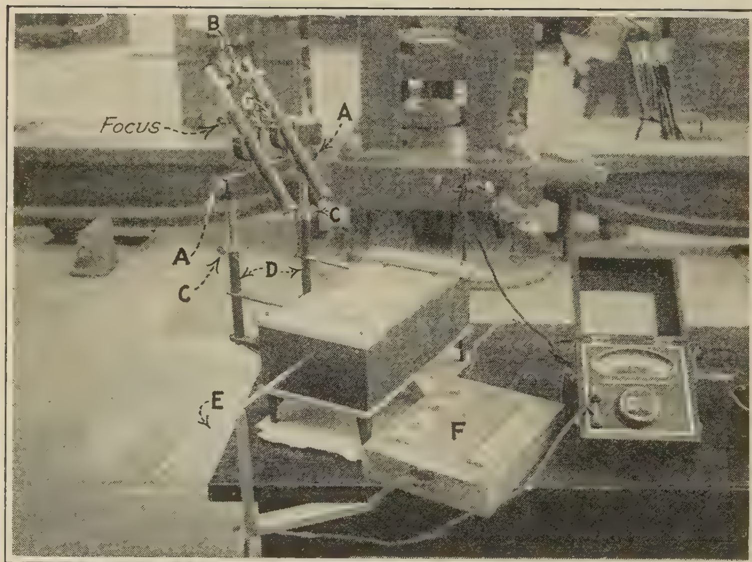
In one end of the furnace is a small hole through which a platinum-rhodium thermo-couple, encased in a quartz glass tubing, was inserted for determining the temperature. This thermo-couple was connected to a portable pyrometer with a calibrated temperature scale reading in degrees *C*.

In order to insure free motion of the expanding specimen, it was placed on two supports on the furnace bottom, one round and one square, placed so as to insure as far as possible an expansion at both ends and also to decrease to a minimum any stress in the specimen due to its own weight. To facilitate the placing of the specimen at the beginning of each test, a plane was established in which the needle points were always placed and parallel to which plane the horizontal cross-hairs were always adjusted.

After allowing sufficient time for the setting of the cement holding the needle points, a specimen was placed in the furnace, the distance between the needle points accurately measured, the telescopes adjusted and the room temperature recorded. The current was then turned on and after a short period of time, to insure equal distribution of heat throughout, various observations were made of the expansion, temperature readings being taken simultaneously with the readings of the two telescopes.

The following table gives the results obtained on five specimens of gunite:

Specimen No.	Original Length, inches	Room Temperature, degrees F.	Highest Temperature Recorded, degrees F.	Difference in Temperature	Mean of Two Telescopic Readings, inches	Coefficient of Expansion per degree F. for Temperature Given
1...........	6.05	57	1098	1041	0.04120	0.00000654
2...........	5.53	60	970	910	0.03220	0.00000644
3...........	5.32	60	1234	1174	0.04020	0.00000643
4...........	6.03	60	1198	1138	0.04390	0.00000641
5...........	5.83	60	1297	1237	0.04637	0.00000643



FURNACE USED IN COEFFICIENT OF EXPANSION TESTS ON GUNITE.

Notes on Concrete Floor Finish.

By E. E. Davis.*

When I was invited a few weeks ago by our secretary and treasurer, Harvey Whipple, to submit a paper on a general topic of "Floors," I hesitated to accept it because it seems to me that the discussion of floors is a subject which covers about as much territory as such subjects as water, transportation and some of the other general topics that we see and read so much about every day.

For instance, floors may be divided into approximately eighteen different groups based on that physical property of the floor which classifies it as an acid-resistant floor, an alkali-resistant floor, as a thermal insulator, as a fire-resistant medium, etc. Then, also, there are some eighteen building materials of major importance used in the construction of various types of floors. Obviously, it would take hours to attempt to give a detailed discussion of floors. We manufacture and install many different kinds of floors, but for the present I will confine this paper to a more or less general discussion of cement floors.

In the first place, there is no question but that the floors in a building play a large part in determining the real value of the building. When one inspects a building, whether it be for a home, a garage, an office, or factory, with the purpose of purchasing or renting it, one of the first things examined on the interior is the floors, and yet I dare say that not until the advent of the American Concrete Institute and the interest shown of late by the Portland Cement Association did any one give the subject of floors anything like the real consideration that it deserves. Good floors are an asset to any building and bad floors are certainly a detriment.

It is generally expected that the roof of a building must be watertight, that the walls, foundations, etc., must be constructed according to the very best engineering practice, and these are usually inspected and approved by the city building departments. It surely would have been to the interest of many owners and tenants had some such building inspection department had something to do with the floor installation; for that part of the building that materially determines its worth, is done in many instances with the most careless and haphazard manner.

Now who is to blame for this, the owner, the architect, the contractor, or the cement worker?

Before answering that question, let us trace the consideration of a floor from its earliest conception in the mind of the architect, through the actual work done by the contractor, down to the final acceptance by the owner.

*E. E. Davis Co., Chicago.

The owner conceives his idea of a floor, he tells the architect who incorporates the owner's ideas with his own experience and writes his specifications for the floor. Later, when it is found that the estimate for the entire building is too high, the owner and architect begin figuring where the expense can be curtailed without, as they think, materially damaging the general scheme of the building. The floor, unfortunately, is one of the first victims of a sharp-pointed pencil. After the contract is let the contractor falls victim to this economical disease and he starts in on the floors. He feels that there is no use of his employing high-class cement finishers at a high overtime hourly rate to finish the floors properly when the owner, by his false idea of economy apparently does not attach much significance to the floors, so long as they are floors. The consequence of this is a poor floor and a decrease in value of the property. The next time the owner puts up a building, or another client consults this architect, cement floors are discouraged and other types of floors are used even though they may not prove as satisfactory as was anticipated. This fault of floor failures is due perhaps to the general apathy of many builders and also no doubt to their lack of knowledge of the many elements which enter into cement floor finish.

Now let us consider the actual construction of cement floors. Many specifications and methods now employed, I think, are wholly wrong, yet they are still quite extensively employed, due usually, I suppose, to tradition, for it seems to me that construction methods are quite generally the legacy of past generations and it is difficult for many to divorce themselves from habit and custom and adopt the more advanced ideas and formulas in their procedure.

There are three main materials that go into the construction of a cement floor, namely, cement, aggregate and water.

Cement.—Cement used should meet the current standard specifications for portland cement adopted by the American Society for Testing Materials.

Water.—Water should be clean, free from oil, acid, alkali and foreign matter. A safe rule for mixing concrete is to use only water that is suitable to drink.

Aggregate.—Now we get to that part of the floor mixture which should require a considerable amount of attention, for, after all, it is the aggregate that must resist the wear. Most of you are familiar with the requirements for aggregates for floor work, as set forth in the Tentative Specifications for Concrete Floors of the American Concrete Institute. I will not take the time to read these specifications, but would suggest that those interested obtain a copy and look them over. There is a very wide range in the quality of aggregates. The sand, I dare say, is dissimilar in every section of the country. I have had a wide experience in using sand in different cities and find that the mixture of cement, the method of mixing, as well as the other aggregates, had to be changed materially to meet each condition. In some localities the sand is entirely unfit for finish cement floor use, due to the high percentage of loam, clay or decayed

vegetable matter. Then, again, to many it may seem that if the sand is clean it is all right, but in addition it must be sharp, irregular in shape, in order to present to its neighbor in the mixture as many points of cleavage as possible. The round silica sand frequently found in lakes and rivers may be faultlessly clean, yet the smooth surface does not lend itself to good cement floor topping. The same general analysis should be given the stone or coarser particles of the mixture.

After selecting the cement, water and aggregate, the next important subject is to determine the mix to be used. This necessarily depends upon the size and nature of the aggregate. The proper mixture to be used can be easily arrived at by using the results of a sieve analysis. I find that very good results can be obtained if we use a mixture of one part cement, one-half part sand, and one and one-half parts clean, coarse aggregate, ranging in size from ¼ to ¾ in. If such an aggregate that is very irregular in shape is used in sufficient quantity to allow a minimum space between each piece and this space filled with sharp, hard particles of sand each of which is coated with a fine film of cement, we will secure a very dense, hard, wearing surface. We, in this manner, employ the cement only for its cementitious value and nothing else. We have experimented with different methods of mixing, using a rotary concrete mixer. We have first thoroughly mixed the water and cement, then deposited the aggregate slowly. In this way I believe we thoroughly hydrate the cement before it comes in contact with the sand and stone so that each particle of cement will function as intended.

Excess water always has its disastrous effects. Excess water in the mixture means voids and minute hair cracks, both of which later contribute to the early disintegration of the floor. After this mixture is deposited on the structural floor slab, it is very necessary to compact it. This is done by rolling, floating and trowelling. By rolling I mean the compacting of the top surface, binding together as tight as possible the entire mass so that the surplus water and foreign matter that might be in the mixture is brought to the surface. This should then be floated with a side float, bringing to the surface a sufficient amount of cement to insure a smooth surface when it is trowelled. The trowelling is not done until the surface has dried to the proper consistency and then it is trowelled three or four times before the surface gets too hard. In this manner a very smooth glassy surface is obtained. Very good results have been secured by grinding the surface instead of trowelling. In other words, the topping is rolled and floated, producing a comparatively level surface which is allowed to harden. It is then ground with a rotary machine equipped with coarse carborundum stones. When the surface is smooth it may be polished with a finer grade carborundum stone similar to the process employed in grinding terrazzo floors.

Even though the best materials be used and the greatest care is exercised in mixing and placing, this all can be undone if the cement finishers are not skilled. I believe of all the work done on a building there is no

part that requires as much skill, patience and experience as trowelling a cement floor. The human element plays a very important part in cement finishing, particularly since the greater part of the actual trowelling is done at night when supervision is harder to maintain and it is vital that patient, honest mechanics be employed who will carry on the work as it should be done.

There are several methods of applying a wearing surface with which all of you no doubt are familiar. The topping can be mixed with the least amount of water possible and applied on the structural concrete slab shortly after it has been poured, or it may be applied later after the concrete has set and hardened. If the latter method is employed, the same or greater care used in mixing, etc., should be given the preparation of the surface to which the topping is to be applied. Another method which we apply very extensively is the application of a dry mixture of cement and aggregate deposited on the structural slab as soon as it is levelled off and while it is still very wet. In this method we depend upon the water in the concrete slab to mix with our dry mixture and hydrate the cement. The thickness of this mixture varies from ¼ to ¾ in., depending largely upon the consistency of the concrete to which it is applied. We believe we can, in this manner, secure a more economical installation and one that will be a true, homogeneous mass and thereby eliminate one of the great causes for failures—the topping cracking and coming loose from the structural slab. We have used all of these methods with a moderate degree of success and prefer the latter because we can go over the work quicker and speed up the job and are certain of securing a surface that will not cause trouble later on.

Another important factor in floor failures is the presence of laitance. In some mixtures and localities this is more apparent than in others and unless some provision is made for its removal, serious results are bound to occur.

I have often been asked why cement finish floors craze and I have said as many times that I did not know. However, crazing is not applicable to floors alone, for we see it on almost every concrete surface. My opinion is that this condition is more apparent where there is an excess amount of water used. I have always insisted that a floor, as soon as it is trowelled, should be covered with sand, sawdust or something of that kind, and then kept for a week or more, depending on the weather. This allows the concrete to dry slowly and thoroughly mature, and, in addition, it protects the surface from abrasion, but I am not so certain but what this contributes somewhat to the crazing.

The matter of air circulation is another item to consider. Obviously, on outside work we usually have more circulation than we need, but on inside work some provision should be made for this.

We hear a great deal about so-called "after treatment" of floors by means of which it is claimed by some that an application of a liquid or other preparation will produce from a disintegrating floor a very hard,

non-abrasive surface. There is no question but that this aids in retarding disintegration of a cement floor, but it is doubtful whether it will entirely correct it. I am not at all trying to discourage the use of a surface treatment for a finish cement floor, for in many cases the building or certain sections of it are used for the manufacturing or storage of chemicals, acids, or other materials having very injurious effect upon the cement, and unless some such treatments are given a floor, its early failure will result. However, I believe it is a serious mistake to attempt to save in the actual building of a cement finish floor with the idea that a surface treatment of some chemical can be applied at a much less cost and will give the same results. There have been offered on the market of late materials that are designed to accelerate the setting and hardening of cement. We have not had a great deal of experience in using these so-called accelerators. We believe that the manufacturers of cement point to the fact that their product sets and hardens in just the right length of time required for it to attain its maximum strength, yet I believe in many instances the use of accelerators is beneficial.

It will not be long, I am sure, before this all important question of floors is given the study and consideration that it really merits and cement floors of varied colors, patterns, designs, etc., will be quite common-place. We must, however, enlist the services of our chemist in doing this, for it is not an easy matter to combine colors with cement and water in such a way that the wearing qualities and strength of the cement are not affected.

The safety factor, that is a non-slip cement surface, is another item that can be greatly improved and while we are now using carborundum with splendid success, yet sidewalks, entrances and outside surfaces of this kind certainly should have some surface treatment which would tend to increase the safety factor.

I will sum up the points in this paper which I wish to emphasize:

1. That the floors in a building structure are a real asset to the value of the property.

2. That there is a great tendency on the part of all concerned to put in floors just as cheaply as possible without regard to their lasting qualities and with little, if any, regard for that branch of the industry in which we are all interested.

3. That in the consideration of floors only the ordinary precaution of proper selections of aggregates, correct mixing, sufficient curing and conscientious finishing are necessary to obtain an acceptable job.

4. That no kind of after treatments will compensate for a poor job.

Central Mixing Plants for the Manufacture of Premixed Concrete.

By W. E. Hart.*

The establishment of central plants for the production of ready-mixed concrete was suggested by the successful operation of similar plants for concrete highway paving. Mixed concrete from such plants is frequently hauled four or five miles with satisfactory results. With this in mind it is natural that progressive material dealers should recognize the possibilities for the sale of ready-mixed concrete, as well as the sale of the raw materials.

Advantages of Central Mixing.—The operation of central plants on large construction jobs brought out a number of advantages which factory production has over hand work and which large permanent plants have over small temporary field plants. Materials can be handled by machinery from the time they leave the cars until they are delivered to the trucks as mixed concrete. This decreases the amount of hand labor required and economizes in the number of times the materials must be handled.

All measurement of materials and operations of the mixer can be handled by one or two men in a properly designed and equipped plant, while several men would be required in the field. The operator soon becomes more skilled and capable of producing a better and more uniform quality of concrete than the average field operator. A properly equipped central plant also is generally capable of measuring materials more accurately and obtaining more uniform proportions than field plants.

Other obvious advantages of a central plant are that it would eliminate the expense of moving field-mixing equipment from one job to another; release the crew required to feed the mixer for use on other work; make the space usually occupied by the mixer and concrete materials available for other uses; and prevent the waste of cement, aggregates and sacks common to field work.

Limitations.—Experience indicates that in the face of these apparent advantages central mixing plants have a number of limitations and disadvantages which are not common to field plants. The use of ready-mixed concrete is limited to work where stiff mixtures can be employed. Concrete of a consistency like that generally used in reinforced work has a tendency to segregate when hauled a few blocks in a motor truck. The water and fine materials float to the top, while the sand and coarse materials compact into a solid mass at the bottom. When the truck is dumped

*Manager, Structural Bureau, Portland Cement Association, Chicago.

the water and fine materials flow out first, while the coarse materials hang in the truck until dug out with picks and shovels. Concrete in this condition is not suitable for use.

A regular schedule of deliveries to a job must be maintained in order to prevent tying up the work and causing loss of time by the contractor's force. This requires an ample number of trucks and the difficulties are increased by traffic delays when the route extends through congested districts with heavy street traffic.

A considerable amount of capital is required to build and properly equip a plant and keep it operating until business can be built up enough to make the operation profitable. The idea is new to most contractors and users of concrete and their prejudices and objections must be overcome before the volume of business can become large enough to show a profit. Contractors have money invested in mixing equipment and would not be justified in discarding their equipment until the success and field of ready-mixed concrete is established.

Personnel and Plant.—The sizes of the crews used at the seven plants studied varied from eight to twelve men with the plants operating at capacity. In the plant operated by eight men the distribution was as follows: One superintendent, one operator for locomotive crane, one mixer operator, one feeding aggregate to bucket conveyor, one measuring aggregates at mixer, one cleaning out material cars and two handling cement.

The distribution of the twelve-man crew is as follows: One superintendent, one mixer operator, one measuring aggregates, two unloading cement from cars, two feeding cement to mixer, one switching cars with tractor, one derrick operator, one signal man, two cleaning cars. The full crew would be required only when the plant was operating at capacity. If operating intermittently a well-designed plant could be handled by three or four men, assuming that one of them was capable of handling a crane, hoist or mixer as occasion demanded. At such times the other members of the full crew could be employed in other work about the material yard.

The plant should be so located, designed and equipped that it can be operated at full capacity with a minimum number of men. As these plants are ordinarily operated to best advantage by building material dealers, the plant should be erected in an established material yard. There should be ample trackage available for cars of sand, stone and cement so that there would be little danger of a shutdown caused by lack of materials. Provision should be made for spotting loaded cars as desired without having to depend on a switching crew or hand-operated pinch bars. Some plants use gasoline tractors for this work, others use a power winch operating a cable direct to the cars or reeved through a system of blocks, while others are fortunate enough to have a locomotive crane or one mounted on crawler treads. Either system is satisfactory and the choice will depend on local conditions and equipment available.

Equipment should be provided to unload material rapidly with a

minimum of manual labor. A derrick or crane equipped with a clamshell bucket is probably the most commonly used. Under certain conditions a belt, bucket, or screw conveyor system can be used to advantage. The success of such a system would usually depend on aggregates being delivered in bottom or side dump cars which could be dumped into pits or underground bins from which the conveyor could deliver the material to the storage bins. Such a system, however, does not permit of a storage pile, and demands a constant supply of cars in order to prevent shutdown.

Cold Weather Precautions.—Plants operating in the northern states should be equipped to furnish warm concrete for cold weather construction. But little additional equipment is required to heat the aggregates and water. An old boiler unsuitable for high pressure work but still satisfactory to furnish steam at low pressure for aggregate bins can be obtained at little expense.

A grillage of steam pipes perforated with ⅛-in. holes laid about 3 ft. above the discharge gates of bins containing fine and coarse aggregates makes a very good arrangement for heating the aggregates. Water can be heated by any one of several schemes commonly used on construction work.

Equipment Capacities and Design.—The mixer should be of such a size that it can deliver 30 cu. yd. or more per hour. This ordinarily requires that the mixer have a capacity of not less than 1 cu. yd. of mixed concrete per batch. One plant in Dallas has a mixer of 2 cu. yd. capacity. Other plants are equipped with two mixers each having a capacity of 1 cu. yd.

The measuring hoppers should be of such design that they will insure accurate measurement of all materials used under all conditions. It should be possible to adjust them quickly for batches of varying proportions as a dozen jobs may be supplied in one day, each requiring a different mix from the others.

Last summer a prominent contractor of Chicago operated a central mixing plant for the construction of a dam which would have been ideal for a plant selling ready-mixed concrete.

One of the most interesting features of this plant was that all materials were measured by weight. A steel hopper having a capacity of 50 cu. ft. of loose materials was mounted under the aggregate bins on the frame of a dial scale. The 30-in. dial of the scale was about 6 ft. back of the hopper in plain view of the operator. The aggregate bins dumped directly into the hopper through metal gates and the cement is fed into it by a screw conveyor leading from the bottom of the cement storage bin about 30 ft. away. One operator measured all materials and fed the one-yard mixer. Two mixers of the same size could have been supplied with ease by this operator.

In charging the hopper, the gate from the stone bin was opened, first allowing stone to flow into the hopper until the pointer on the scale reached the mark for stone posted on the dial. As the pointer approached the

mark the flow was gradually cut off and stopped just as the pointer reached the mark. When the sand had been weighed out on top of the stone, the motor driving the screw conveyor was started by a controller mounted near the side of the hopper. When the required cement had been weighed the motor was stopped, and contents of the hopper discharged into the mixer through a gate in the bottom.

Materials Measurement.—The measurement of materials by weight has several advantages over measurement by volume. One advantage is flexibility. No adjustment of the hopper or other equipment is necessary to change the proportions. The different proportions were used on this job and markers in three colors were posted on the dial to show the weights of materials required for each. That is, a 1 : 2 : 4 mix was indicated by red markers, another mix was indicated by green, and another by black. The operator changed proportions by merely using a different set of markers. Complicated proportions such as 1: 1.3: 3.4 could be measured as easily as more common proportions. A complete list of proportions with weights for each printed on a convenient card would enable the operator to deliver without delay any mix desired.

Sand Bulking Errors.—Another advantage of weighing equipment is its great accuracy. When water is added to dry sand it swells, sometimes as much as 30 per cent. That is, if a cubic foot of dry sand weighing 100 lb. were mixed with 6 lb. of water, the volume of the sand and water may be increased to 1.3 cu. ft. A cubic foot of this moistened sand would then weigh only 81.5 lb. If the measuring hopper was set to measure 12 cu. ft. of sand it would actually measure only 980 lb. of this moist sand instead of 1200 lb. as intended. In other words, each batch of concrete would actually lack more than 2 cu. ft. of sand on account of the bulking or swelling of the sand due to moisture. Concrete deficient in sand is richer in cement than intended and costs more per cubic yard than it should.

The variation of sand content in concrete and the errors caused by bulking can be largely eliminated if the sand is measured by weight instead of by volume. In order to insure greater uniformity in the quality and proportions of concrete batches the Iowa State Highway Commission now requires that all aggregates, both fine and coarse, used in state highways be measured by weight. A correction for the estimated moisture content added on the scales practically eliminates errors in proportions due to bulking and moisture content of the sand.

Bulk cement is more difficult to measure accurately by volume than damp sand. The weight of a cubic foot of cement will vary as much as 25 per cent with the manner and degree of compacting. It can easily be measured by weight, as 94 lb. of cement is universally considered as one cubic foot.

Advantages of Bulk Cement.—Bulk cement has a number of advantages over sacked cement at permanent central mixing plants. It eliminates the work and expense of emptying, cleaning, baling and shipping empty

bags. It can be unloaded from cars into bins and handled from bins to mixer by machinery to better advantage than cement in bags. One of the most successful methods for unloading it is by means of a power scoop operated by two men with a small electric or gasoline hoist. One man inside the car drags the scoop back from the door, pushes it into the cement and guides it while the other man outside pulls the loaded scoop to the door with the small hoisting motor and a line running through a block at or near the door. When the loaded scoop reaches the door it is dumped into the bin or into a small pit whence the cement is carried to a bin by a conveyor. One large user of bulk cement finds that three men can unload three cars of bulk cement per day with this outfit.

When the cement storage bin is at one side of the material bins it will usually be necessary to deliver the cement from the bin to the measuring hoppers by means of a belt or screw conveyor. This conveyor should be driven through a clutch or electric motor so that it can be started quickly and stopped when the amount of cement required for a batch has been delivered to the hoppers.

Aggregate Storage and Water Control.—Most storage bins for aggregate in plants now operating provide storage capacity of materials sufficient for three or four hours capacity of the mixer plant. These bins are most conveniently located over the measuring hoppers so that materials can be drawn directly from the bins into the hoppers by gravity. When two or three grades of coarse aggregate are used it may be advisable to provide an overhead bin for each.

An accurate control of the water content for each batch is essential to the uniform consistency and necessary to successful hauling of mixed concrete. A water-measuring tank capable of quick adjustment for the varying amounts of water required should be provided at the mixer.

Concrete Delivery.—Mixed concrete has been delivered successfully in large trucks on solid tires. Where volumes are large, the roads well paved, and the hauls short, the large trucks will deliver concrete at a lower cost per ton-mile than the light trucks. When conditions are reversed the light trucks will usually have an advantage. Pneumatic tires under heavy loads cost more per mile of operation than solid tires, but they cause less vibration and wear of the truck mechanism and less compacting of the concrete than solid tires. Trucks on pneumatic tires can make better speed and therefore longer hauls than solid-tired trucks. Some central mixing plants do not own all their trucks but contract for the delivery of the concrete with local trucking concerns, when extra trucks are required. This relieves the plant operator of the investment in a large fleet of trucks and the responsibility of their care and operation.

Tests.—Experiments and tests carried out by the Bureau of Public Roads in 1921 show that the strength of concrete is not impaired when hauled for periods as great as three hours. These tests show that there is no danger of injuring concrete by hauling it for any economical distance, provided it is not segregated when deposited in its final position.

These tests and field experience indicate that concrete compacts into a solid mass in the truck body when it is hauled for a considerable time, and that the rapidity and degree of compacting is greater with concrete of thin consistency than with concrete of a stiff consistency. Concrete to be hauled more than a few blocks should be of such a consistency that it will show a slump of not more than 4 in. when tested with the slump cone. Experience shows that concrete which has been thoroughly mixed does not segregate as quickly as that which has been imperfectly mixed. A minimum of 1½ min. mixing time is recommended.

Emptying Compacted Concrete.—When a load of concrete is compacted in a truck body it can be dumped only with difficulty unless special means are provided to clear the body when the load is dumped. Several schemes have been used to overcome this difficulty. In one method a piece of log chain is laid across the front end of the body with the ends laid out along the bottom toward the rear end. If the load does not clear itself from the truck when dumped a pull on one or both ends of the chain will usually clear the truck. Another plan is to provide a false bottom in the truck of canvas or sheet metal which slides 2 or 3 ft. when the load is dumped. There is seldom need for such equipment, however, if the concrete is mixed for at least 1½ min. and has a slump within specified limits.

Field for Central Plants.—The application of the central mixing plant to commercial business must be confined to certain definite limits. Present-day knowledge sets as a limit for such work a concrete with a slump not greater than 4 in. The field for such plants is therefore limited to pavements, curbs and gutter foundations, plain concrete floors, retaining walls and structural concrete, with a sufficiently large section to permit the placing of concrete around the reinforcement with a reasonable amount of spading. Concrete should always be dumped from the truck into a batch box. Under no circumstances should delivery be made directly into the forms. Concrete of the required consistency when placed directly in the forms cannot be spaded or spread properly without honeycombs and the exposing of reinforcing bars. Another feature that must be guarded against is the remixing of the concrete on the job by adding additional water. Such a procedure will unquestionably reduce the strength of the concrete and lead to a practice on the job that will place the central mixing plant in disrepute.

Advantages and Disadvantages Summarized.—In conclusion, permit me to sketch the advantages and disadvantages of the central mixing plant. One of the principal advantages of these plants is the fact that it places the mixing of the concrete in the hands of one man, where materials can be graded, proportioned and mixed with greater uniformity than in the field. Manual labor is reduced and machinery substituted therefor. The expense of moving contractors' equipment from place to place is eliminated and at the same time storage space for materials is eliminated on the job. Perhaps the greatest advantage in the central mixing plant is the fact that only the stiffer mixes can be transported. If the idea of the central

mixing plant becomes established, we are sure of reducing very materially the water content in all concrete.

The principal limitation of the central mixing plant is one of transportation. Only the stiffer consistencies can be successfully delivered by truck without segregation and settlement into the body of the truck. Other difficulties to be overcome are the transportation of concrete into congested areas and the necessity of avoiding delays in the transportation of these materials to a large construction job. At the same time a great deal of prejudice will be set up by the contractors, due to the fact that they already have a great deal of expensive equipment on hand which cannot be discarded, due to the character of their business. No doubt there will be a certain amount of prejudice in the minds of officials to the exact results to be obtained by a central mixing plant. They may not be entirely in sympathy with laboratory tests and for that reason will rule out the use of central mixed concrete, fearing that the ultimate strength of such concrete will be impaired by transportation. This, however, is a minor point, due to the fact that mixed concrete has been transported by a number of means for quite a number of years.

Generally speaking, a central mixing plant is entirely practical and safe, providing it is in the hands of competent men. The same axiom applies to the central mixing plant as to any other class of work. It is not "the use but the abuse" of a system that causes the difficulty.

DISCUSSION.

C. B. FOSTER.—Where did you start? Mr. Foster.

W. E. HART.—The projects I have been on had the plant located in the center of the job. Concrete is placed from a point two miles north to the plant and then two miles south to the plant. By the time the north part is cured and ready for traffic, concrete is then placed two miles north of the point of beginning of the first piece. Mr. Hart.

MR. FOSTER.—Then they have this mud road to travel over with their loaded trucks in the wet weather? Mr. Foster.

MR. HART.—That is true. Mr. Hart.

MR. FOSTER.—You spoke of a 1½-min. mix, and just the other day there was a man spoke of a 1-min. mix. I have had experience with concrete nearly ever since I started, and I find that I can get as good a mix in some mixers in thirty seconds as I can get in others in five minutes, and I think that is a point we ought to give consideration to. I do not think the time has anything to do with it. Mr. Foster.

MR. HART.—The reason we have mentioned a 1½-min. mix is because it is possible for the plant to mix a minute and a half, if they do not have a string of trucks waiting. Longer mixing is desirable, because it aids materially in the removal of the concrete from the body of the truck. Mr. Hart.

H. F. FAULKNER.—Have you ever considered the difference in the use of flat and V-shaped trucks for delivery? Also, could not concrete be delivered into hoppers through a spout as a remixing plant, then into buckets or barrels to be delivered into the elevator? In that way you might overcome segregation. Mr. Faulkner.

MR. HART.—In answer to the first question, I have had experience in both cases. On one particular job we used fourteen trucks, seven of them having one-yard V-shaped bodies, while the others were 3½-ton trucks with flat bodies. In both cases we had some trouble if the mix was too wet. On this particular job they were using a slump of about an inch and a half with the result that it came out very easily. If they got the mix too wet they had more difficulty in the flat-bottom than in the V-shaped truck body. In the second case, I can see no objection to the instance you mention, if they do not add water when they remix. Mr. Hart.

E. LICHTENBERG.—It may be interesting to the members of the Institute and Mr. Hart, that about eight or ten years ago we actually constructed a device that I believe would overcome all the objections that Mr. Hart brought up to the central mixing plant. This was a mixer mounted on a truck, that is a 5-ton chassis was used and a mixing drum was mounted on the truck instead of the regular body. This cylindrical drum had a capacity of about 2 cu. yd. dry material. The cement had a separate compartment and the water tank was mounted over the drum. Mr. Lichtenberg.

The truck was charged at a central proportioning plant and then hauled to the job. About two or three minutes before the driver got to the job he threw in a clutch lever and started the mixing drum to work and when he arrived on the job, the mix was all mixed and ready to dump right into the forms. We actually constructed this machine, but before we completely finished the job we realized that the whole scheme was premature. From a commercial standpoint there was no call for such a device, because the state of the art had not advanced as much as it has today, but the scheme is still in existence and is wholly practical and it may eliminate all your troubles about hauling mixed concrete.

Mr. Wilson. J. P. WILSON.—Two years ago the city of Grand Rapids paved some street where the material was mixed at a central mixing plant and hauled as far as 3½ miles. Sometimes the material was held at least an hour before it was dumped. They took test pieces at the mixing plant and test spots were marked in the street and cores were taken from the street to correspond with the cylinders stored at the central mixing plant. They sent those cylinders down to me at Detroit to test, and we found that the cores taken from the street showed 10 to 20 per cent higher strength than we got out of the laboratory cores taken at the central mixing plant and stored according to the best practice. We got concrete from that street as high as 6,200 lb. to the square inch. It had been hauled three miles and a half, and some of it held at least an hour in the trucks before it was dumped.

Finishes in Stucco.

By Samuel Warren.*

The relationship of stucco finishes to the house on which they are applied presents many interesting possibilities. Unlike practically all other building materials, stucco is not a series of complete units fitted together on the job, but is, until the moment of its application, a shapeless flowing mass, capable of endless possibilities. Brick, wood or stone are definite in color and texture. Their appearance in the finished work can be readily judged before they are in place and at the same time offer little latitude after the choice of some particular type has been selected.

Stucco is vastly different. One mix of definite proportions gives many and varied effects, according to its manipulation. It is the object of this paper to bring out some of the methods used in securing these various effects.

We speak of color and texture in stucco. This is not an accurate statement, for color and texture are almost synonymous terms. We can secure an effect of so-called texture on a perfectly smooth surface, or we can take a material of uniform color and by giving it a rough surface secure several different shades. For instance, we can take a stucco, say composed of one part white portland cement, three parts yellow sand, and a little lime and by different manipulation, give it many different shades. A smooth troweled surface will give a very light buff. A surface slightly torn with the trowel will introduce lights and shadows and deepen the color. A surface rubbed with a carpet float or brush after it has been on for a few hours will show a very distinct and deeper shade of yellowish buff. We can therefore control the color of our stucco, not only by the inherent quality of the color of the mix itself, but by the method of its application.

These combinations of color and texture mean much to the portland cement industry in that they offer in portland cement stucco a material of almost unlimited possibilities and a consequent elimination of monotony. This is a very important point when the growing use of portland cement stucco is realized, for people do like, at least, a touch of individuality in their houses. The old day when rows of identical houses were built is fast passing, and even though some real estate development may, in the interest of economy, keep the houses practically alike, yet it is possible by slight structural changes and by variation in the stucco itself, to avoid monotony and yet harmonize all the buildings.

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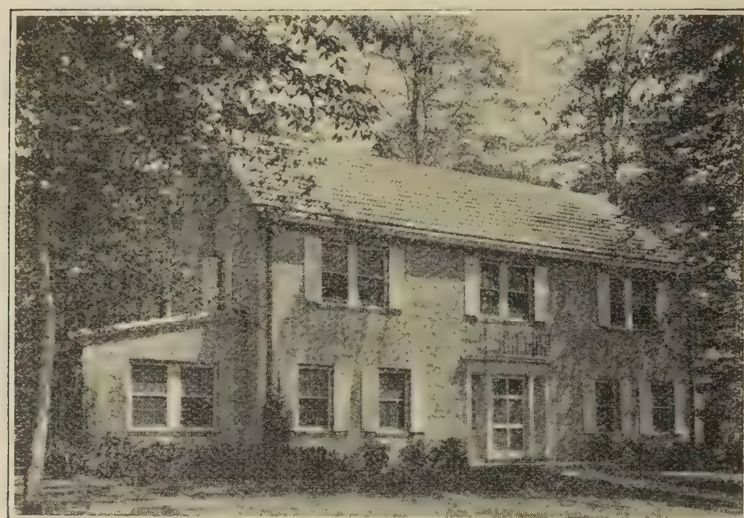
*Manager, Atlas-White Dept., Atlas Portland Cement Co., New York City.

There is also a relationship between the various stucco finishes and the type of the house architecturally. Most houses follow certain principles of architecture and the stucco should be in keeping with them. For instance, in the old days in the southwest, most construction was of adobe block and adobe stucco. The adobe stucco was applied by hand and worked into an approximately smooth finish by hand. It is therefore in keeping with this type of architecture to approximate as far as possible the finish on these adobe buildings. Again we have colonial architecture with many distinct variations. In its earlier days, particularly in New Jersey and Pennsylvania, the detail was often very simple. The stucco was used for



Kenneth Dalzell, Architect.
STUCCO AND TILE IN A MAPLEWOOD, NEW JERSEY, SETTING.

the purpose of sealing the "weather" sides of the rough laid stone walls of the building, usually the north and the east, from the driving rains and cold winds. The stone was usually rough and the stucco followed the contours of the stone.

As the colonial architecture became more formal and possessed of great perfection of detail, we find the stucco also being put on with great care to insure uniformity. Lack of finish elsewhere in the house calls for lack of finish in the stucco. Refined detail of the trim must not clash with a rough textural stucco.

The old English cottage type of architecture was in many senses an uncertain type. It has certain characteristics but allows for a great latitude in treatment. Its roofs, high pitched due to the use of thatch, are well known. These houses were undoubtedly built in the days when the

J. K. Branner, Architect

RESIDENCE OF CHARLES L. LEWIS, SAN FRANCISCO, CALIF.

A. J. Thomas, Architect.
HOME OF FRANCIS KEIL, SCARSDALE, N. Y.

same group of men built the foundations, laid the brick, fashioned the floor timbers and floors, and applied the stucco filling between the half timber. The solidity of their work is proven by its ability to last. Their stucco stayed on but without the slickness of finish of modern workmanship.

In modern days of machine-made materials, there is an attempt to approach, in handwork, the slickness and uniformity of the machine.

Julius Gregory, Architect.
ENGLISH EFFECT IN A PELHAM, N. Y., HOME.

Today our timber comes to the job (supposedly at least) to perfect dimension, sawed irrespective of irregularities of grain. In the old days the tree was felled and squared by axe or adze and usually the wind of the tree was retained.

There is also a peculiar satisfaction in hand workmanship. A piece of timber dressed four sides in a planing mill is like every other piece—

Bloodgood Tuttle, Architect.
HOME OF MARY M'KELVEY, SPUYTEN DUYVIL, N. Y.

one look at it and we realize nothing back of it but the machine. We see an adzed timber and our mind is taken back to the man who hewed the log. So in stucco there are many possibilities of craftsmanship but it must be true craftsmanship and not imitation. If we take a log of wood which has already been through the saw mill and try to make it have the appearance of being adzed the result is quite sure to be a poor imitation. It is not honest. The same applies to stucco. These various textural finishes should show the method that the mason used in getting the stucco onto the wall and not some irregularity worked into the surface after the stucco is on. If that thought be kept in mind, it will be found much simpler to secure interesting effects.

The degree of texture must also be considered. If the building is near the road, so that it is usually seen from a distance of only a few feet, the texture should be only slightly rough. Otherwise it will appear too coarse. Again, if the house stands back among the trees, and the usual viewpoint is several hundred feet, the scale of the texture should be correspondingly increased. Always lay up a good sized sample, say 6 ft. square, and then examine it from various distances and various angles to judge its fitness for the given work.

Coming to more particular points, the degree of plasticity of the mix has much to do with the finish itself. If a poorly graded sand—one in which the particles are much the same in size—is used, it will be found difficult to secure an interesting effect, as the material is dead and works badly under the trowel. A well-graded sand is fatty and works with freedom and easy control under the trowel.

Suction of the wall must also be watched, for if the wall is too dry the absorption of the water from the finish coat will give the finish an almost instant set. The wall should be wetted so that the stucco maintains its full plasticity for at least half an hour. This usually means at least two thorough drenchings of the wall the previous day and again just before the work is done. As it is sometimes difficult to cover every square inch of the base coat in applying these finishes, most masons apply a very thin cover coat over the base coats and into this work the true finish coat. This, of course, is only applied slightly ahead so that it does not have a chance to harden.

It may be felt that it is difficult to teach the ordinary plasterer, who is used to perfection of surface, the method of applying these finishes, but it has been my experience that most of them take to them very readily once they grasp the idea.

Report of Tests Made to Determine the Temperatures in Reinforced-Concrete Chimney Shells.

By E. A. Dockstader.*

Reinforced-concrete chimneys have been in use in this country for about twenty-five years. With the growth of central power stations, large reinforced-concrete chimneys have become increasingly common. This type of construction has many features in its favor particularly for chimneys superimposed on the power station structure where the lighter weight of a reinforced-concrete chimney as compared with a brick chimney is important. Cracks, which have appeared in a number of reinforced-concrete chimneys, have led to some doubt on the part of engineers as to whether such construction is reliable. Such cracks can usually be traced to improper design, faulty workmanship or a combination of the two. It was with a view toward improving the design of these structures that the tests under discussion were made.

Theoretical analyses of the stresses in chimney shells due to the weight of the chimney and the force of the wind upon it are available. Wind velocities have been measured for many years with more or less accuracy and records kept of maximum conditions. It has, therefore, been possible to proceed with reasonable accuracy in designing reinforced-concrete chimneys to resist the stresses due to dead-load and wind, and for many years the emphasis in the design of chimneys has been placed on the proper proportions of shell thickness and reinforcement to resist these stresses.

It has long been recognized that the stresses in the chimney shell due to temperature were of great importance and theoretical analyses of these stresses, both circumferential and vertical have been developed. The actual temperatures in the concrete of the chimney shell under actual operating conditions have not heretofore been measured. It has been necessary, therefore, to assume the difference in temperature in the concrete between the inner and outer faces of the chimney shell, in order to use the formulas resulting from theoretical analyses. This indeterminate factor has prevented the application of these formulas to the design of reinforced-concrete chimneys with any degree of assurance.

During the summer of 1922 it was suggested by the Heine Chimney Co., of Chicago, Ill., that provision be made in the construction of a chimney which they were about to erect for the New Bedford Gas and Edison Light Co., under the direction of Stone & Webster, Inc., so that tests could be made when the chimney was in service to determine the temperatures in the concrete of the chimney shell. The New Bedford Gas and Edison Light Co. was not only willing that the tests should be made but during their progress co-operated in every way and rendered much valuable

*Stone & Webster, Inc.

assistance. The actual tests were planned and carried out by Stone & Webster, Inc.

The chimney selected for test was erected in the fall of 1922. It is 15 ft. internal diameter at the top, and 188 ft. high above its foundation,

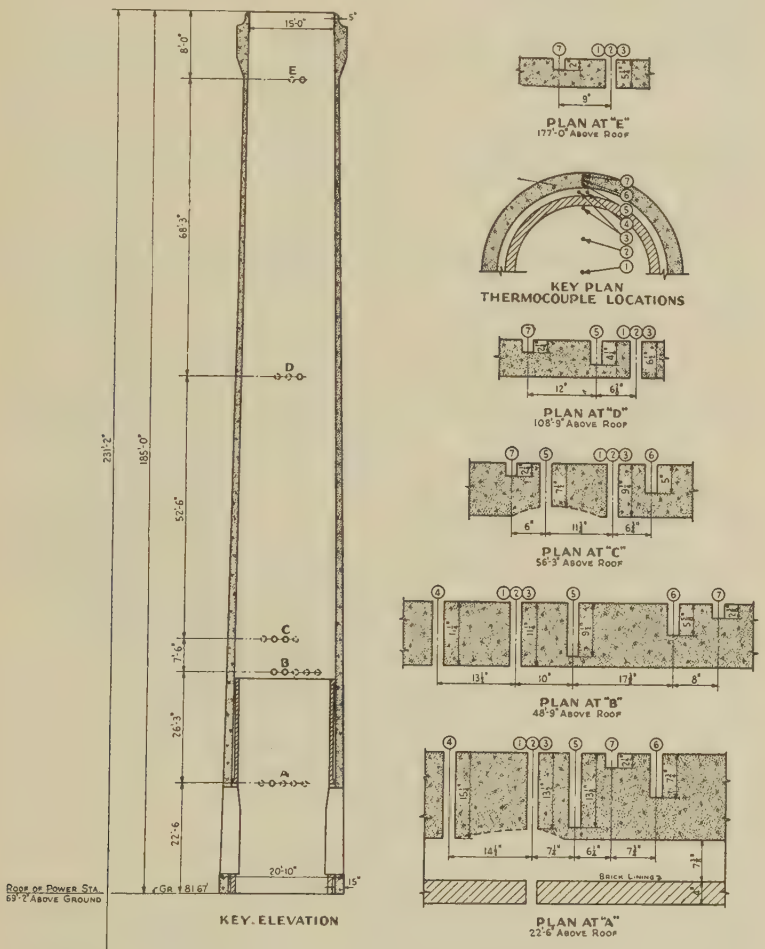


FIG. 1.—DETAILS OF CHIMNEY, SLEEVES AND THERMOCOUPLE LOCATIONS.

which foundation is superimposed on the steel framing of the power station above the boilers. A photograph of the construction drawing of the chimney is attached. The tests were made on the north chimney of the two shown on this drawing. This chimney is designed to serve three 1,608-hp. Babcock & Wilcox boilers operating at 250 per cent rating,

although at present only two of these boilers are installed. The flue from each boiler enters on opposite sides of the chimney. A steel plate baffle in the chimney extending 6 in. above the top of the flue opening separates the opposing streams of incoming gases. The boilers are coal fired through

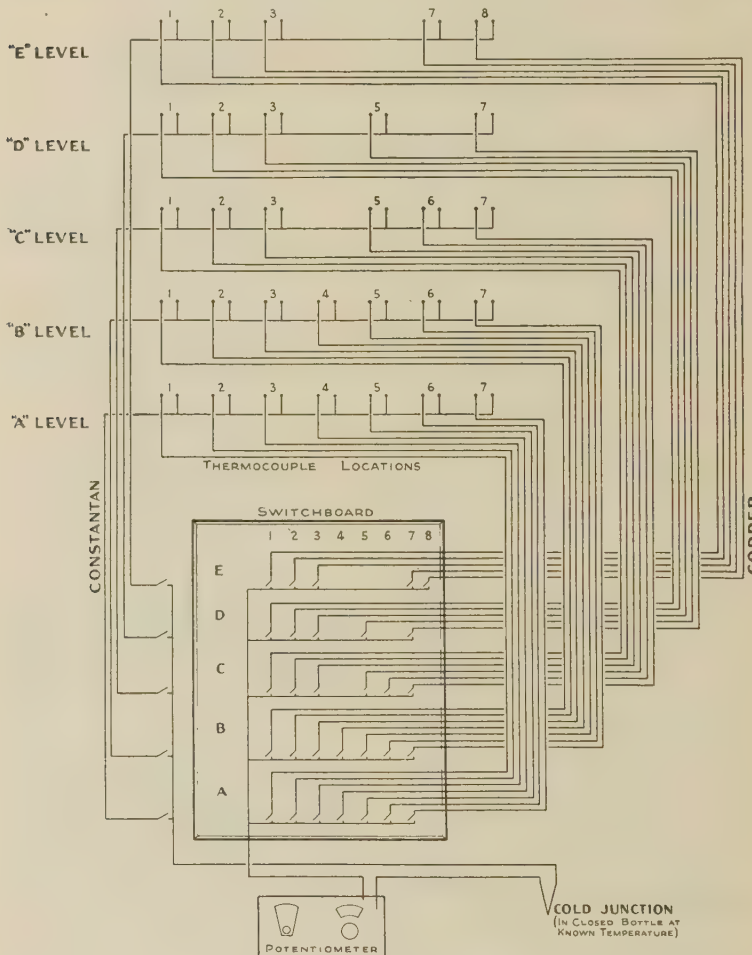


FIG. 2.—WIRING DIAGRAM.

Frederick underfeed, multiple-retort stokers under forced draft. There are no economizers or induced draft equipment.

The chimney is lined with common brick to a point approximately 2 ft. below level "B", Fig. 1. This brick lining is 8 in. thick to a point approximately 6 in. below level "A", Fig. 1, and 4 in. thick above this

point, with an air space between the brick lining and the concrete shell, this air space being sealed at the top of the brick lining.

When the chimney was constructed sleeves of extra heavy brass, 2¼ in. inside diameter and approximately ¼ in. thick, were cast in the chimney shell at various levels as shown in Fig. 1. The sleeves were placed on the south side of the chimney in all cases. It was the intention to arrange these sleeves so that at each level the temperatures could be measured (1) of the gases at the center of the chimney, (2) of the gases midway between the center and the chimney shell, (3) of the gases immediately adjacent to the chimney shell, (4) of the air (where the chimney was lined) between the lining and the concrete, (5) of the concrete approximately 2 in. from the inside of the shell, (6) of the concrete at the center of the shell, and (7) of the concrete approximately 2 in. from the outside of the shell. It should be noted that in plotting and tabulating the results of the tests, the readings are referred to throughout as numbered above, so that for example, point (1) always refers to the temperature of the gas at the center of the chimney, point (5) the temperature of the concrete nearest the inside of the chimney shell, point (7) the temperature of the concrete nearest the outside of the chimney shell. Where readings were not taken at a given point, as for example point (4) between the shell and brick lining for levels above the lining, point (5) at level "C", and points (5) and (6) at level "E", these numbers are omitted in the plotted and tabulated results to maintain the identification of a given number with a point in the same relative location. At level "C", readings at point (5) in the concrete nearest the inside of the shell could not be obtained due to a bulging of the form so that this sleeve projected clear through the concrete.

Readings were taken by means of "Copper-Constantan" thermocouples. Fig. 2 shows the wiring diagram for these couples, one couple being installed at each location so that readings could be taken at any point by merely closing the proper switch on the switchboard inside the power station. Couples in the gas stream were inserted through iron pipes to hold them in their proper location, the couple being insulated from the pipe. For readings in the concrete, small holes were drilled directly into the concrete, the "hot junction" of the thermocouple inserted and the entire sleeve filled solid with plaster of paris around the porcelain tubes which insulated the connecting wires. Independent copper leads were run from each thermocouple to the potentiometer switchboard. Originally a single bare Constantan wire was used, common to couples at all levels but preliminary readings showed that this gave very inaccurate results. The single Constantan wire was therefore removed and individual insulated Constantan leads were run for the couples at each level, as indicated in Fig. 2. Readings were taken by a potentiometer in milli-volts. The copper Constantan wire used was calibrated at the Massachusetts Institute of Technology and a table prepared from which the millivolt readings were converted to degrees Fahrenheit for plotting and tabulating the results of the tests.

Outside air temperatures were taken by means of a thermocouple mounted at level "E" and a thermometer on the roof near the base of the chimney.

A tabulation of all of the readings taken after the apparatus was installed and tested follows:

TABULATION OF TEMPERATURE READINGS.

Date	January 25, 1924					January 28, 1924		February 26, 1924		Mar. 19, 1924	June 18, 1924
	A.M.	A.M.	P.M.	P.M.	P.M.	P.M.	P.M.	P.M.	P.M.	P.M.	P.M.
Time started	9.30	9.50	2.56	3.15	4.40	2.20	3.46	1.30	3.15	2.30	3.00
Time finished	9.44	9.58	3.05	3.30	4.55	2.32	3.58	1.48	3.30	2.45	3.40
Temperature in Degrees Fahr.:											
Thermocouple E1	385	382	392	391	388	359	362	367	374	402	424
E2	387	387	390	385	385	352	356	367	370	390	410
E3	308	314	325	321	315	287	296	307	309	342	348
E7	99	108	137	137	130	123	108	143	138	158	166
E8 (outside air)	52	49	60	58	48	33	31	54	52	62	79
Thermocouple D1	412	410	410	411	405	376	377	388	394	422	439
D2	393	391	400	397	394	364	363	374	380	402	424
D3	218	219	266	247	244	203	197	232	228	251	284
D5	102	108	123	125	130	131	123	144	147	169	182
D7	92	97	118	118	123	115	107	138	135	152	162
Thermocouple C1	419	415	421	420	417	383	388	396	400	427	450
C2	418	413	421	421	416	383	386	396	400	427	442
C3	284	285	325	317	312	269	266	268	267	283	420
C6	122	125	146	145	149	143	143	163	162	183	210
C7	101	107	126	126	128	115	103	142	126	141	166
Thermocouple B1	423	421	425	422	422	390	392	401	401	430	454
B2	424	421	424	423	421	390	390	401	400	430	454
B3	396	393	400	399	400	363	363	374	378	409	442
B5	109	108	134	135	137	113	113	126	126	144	162
B6	142	145	170	172	179	170	169	188	188	212	229
B7	108	111	118	120	117	121	115	134	134	154	178
Thermocouple A1	452	448	435	440	445	421	414	418	420	455	469
A2	430	434	428	422	429	395	390	396	400	436	462
A3	429	432	425	427	423	383	379	388	392	429	442
A4	206	208	220	220	224	190	195	209	214	238	263
A5	124	122	134	133	139	112	112	130	134	156	174
A6	90	90	99	99	101	80	82	100	102	121	142
A7	76	75	78	77	77	64	63	78	78	92	114
Air temperature at base of chimney by thermometer	...	...	44	...	...	21	21	40	38	38	86
Weather	...	...	Clear	...	...	Clear		Medium	Clear	Hazy	Clear
Wind	Northwest—Velocity estimated 10 mi. per hr.					Strong and puffy		Moderate		Light	Note A
Rated boiler horse power on chimney	...	...	3216	...	...	3216	3216	3216	3216	3216	3216
Boilers operating at rating of	...	...	210%	...	...	205%	220%	203%	198%	209%	205%
Total boiler horse power on chimney	...	...	6754	...	...	6593	7075	6528	6368	6721	6593
Average CO_2 of chimney gas at "B" level by volume	...	...	9.5%	...	...	8.9%	9%	9.5%	9.4%	9.75%	10.5%
Draft in inches of water at base of chimney	...	...	1.10	...	...	1.23	...	1.05	1.18	1.10	1.0
Draft in inches of water at "B" level	...	...	.93	...	...	.84	.92	...	.88	.80	...

NOTE A.—Wind velocity 300 to 500 ft. per min. by anemometer.

NOTE B.—Thermocouples E3, D3, C3, and B3 were located approximately 4 inches from the inside of the chimney shell and A3, approximately 4 in. from the inside of the brick lining.

The tabulated draft readings were taken with a differential draft gauge. Gas samples were taken at the time of the tests by means of an Orsat apparatus. Samples taken on Jan. 25, 1924, at level "B" showed:

$CO_2 = 9\frac{1}{2}$ per cent by volume.

$O_2 = 9\frac{1}{2}$ per cent by volume.

Gas samples taken at levels "B" and "D" and at various points in the chimney from its center to within 1 in. of the concrete shell showed no measurable difference in CO_2 apparently indicating uniform gas distribution throughout the chimney both vertically and in cross section.

The average temperature of the gases at the boiler uptakes on Jan. 25, 1924, was 475 deg. F., the computed mean velocity of the gases 775 ft. per minute, and the average boiler efficiency 75 per cent.

The coal burned throughout the tests was that known as "New River," having an average heat value of 14,400 B. t. u. per pound. No analysis was made of the coal actually burned at the time of the tests but a typical analysis of this coal shows:

	Per Cent
Carbon	83.74
Hydrogen	4.26
Oxygen	2.47
Nitrogen	1.51
Sulphur	.66
Ash	7.36
	100.00

It will be noted from the reading taken Jan. 28, and Feb. 26, that in both cases the readings taken later in the afternoon show higher gas temperatures and lower concrete temperatures at point 7 than the readings taken earlier the same afternoon while the outside air temperatures and wind conditions remained practically unchanged. The thermocouples were all located on the south side of the chimney and point 7 is nearest the outside of the concrete shell. It is possible that the shift in the rays of the afternoon sun may have lowered the concrete temperatures at this point to some extent.

In Fig. 3 the average of the three sets of afternoon readings on Jan. 25, the average of the two sets of readings on Jan. 28, and the readings of June 18 have been plotted from the center of the chimney to the outside at each level. It should be noted that the curves of this diagram do not represent the temperature gradient from the center of the chimney to the outside but are made up by merely plotting the observed temperature at the points where readings were taken and joining these points by straight lines. It is a well-known fact that at the surface of a heat barrier there is a drop in temperature in the direction of the heat flow so that at the inside of the chimney shell there is undoubtedly a considerable drop from the temperature of the gas impinging against the concrete to

the temperature of the concrete itself at its inner surface. The probable character of the temperature gradient is illustrated in Fig. 5 with abrupt drops in temperature at the surfaces of the concrete rather than straight lines between observed points, although, of course, the exact nature of this curve is somewhat conjectural. Fig. 5 is not intended to indicate the actual temperature gradient but merely its character as the curves of the diagram were plotted from a single set of readings.

The readings at level "B" seem inconsistent. The actual readings were checked several times and new thermocouples substituted to assure

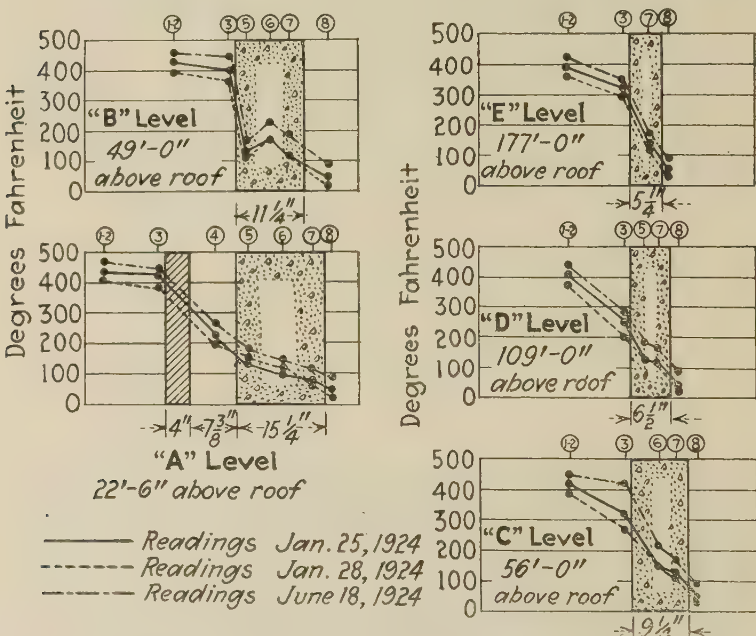


FIG. 3.—RECORD OF TEMPERATURE READINGS PLOTTED ON RADIAL LINES THROUGH CHIMNEY SHELL AT VARIOUS LEVELS.

their accuracy. It will be noted that level "B" is just above the top of the brick lining. It may be that the flow of the gases is so directed that they do not strike directly against the concrete at level "B" or that some structural detail at the thermocouple locations, such as the possible close proximity of steel reinforcing to points 6 and 7 accounts for the discrepancy in readings at this level.

In Fig. 4 the same readings plotted in Fig. 3 are again plotted, but in this diagram the readings of a given point relative to the chimney shell are plotted at the various levels throughout the height of the chimney. In general the gases become cooler toward the top of the chimney,

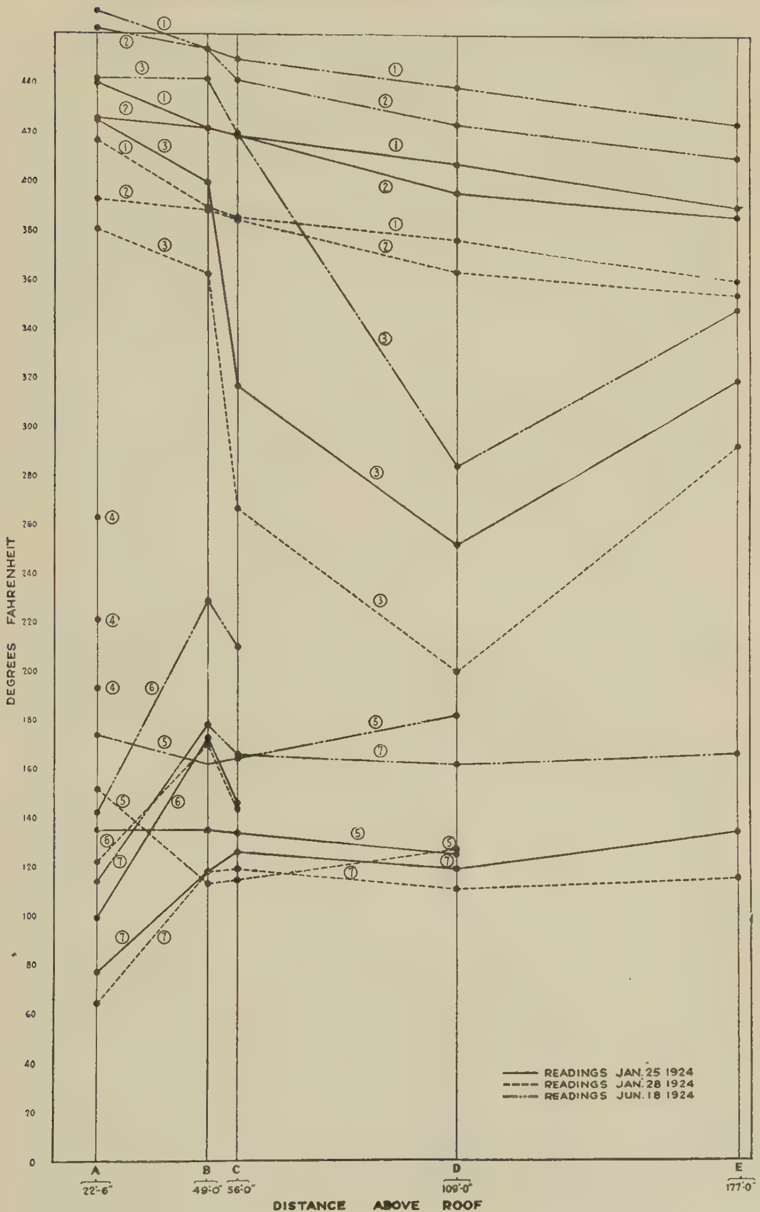


FIG. 4.—RECORD OF TEMPERATURE READINGS PLOTTED ON VERTICAL LINES PARALLEL TO CHIMNEY SHAFT.

as would be expected. The drop in temperature at point 3 at level "D" is probably due to the junction of the thermocouple being located closer to the chimney shell at this level than at other levels.

The maximum stress due to temperature to be provided for in the design of the chimney is occasioned by the greatest difference in temperature in the concrete itself between its inner surface and the temperature reinforcement. The readings perhaps do not give this maximum condition of temperature difference in the concrete which may occur on a very cold winter day when the boilers are first put on the line after being banked over a low station load period. The hot gases impinging suddenly on the inner surfaces of the concrete may raise its temperature rapidly, the outer surface becoming warm gradually as the heat flows through the concrete, thereby giving a greater temperature difference and consequent greater stress. The wiring has been left in place and it is hoped that during the coming winter further readings bearing on this question may be obtained.

The following is a tabulation of the temperature differences in the concrete of the chimney shell shown by the readings:

TABULATION OF TEMPERATURE DIFFERENCES IN CHIMNEY SHELL.

Radial Distance from Readings	D-5 to D-7		C-6 to C-7		B-6 to B-7		A-5 to A-6		A-6 to A-7		A-5 to A-7	
	2 Inches		2¾ Inches		3¼ Inches		5⅜ Inches		5¼ Inches		10⅝ Inches	
	Total Temp. Diff.	Temp. Diff. per Inch	Total Temp. Diff.	Temp. Diff. per Inch	Total Temp. Diff.	Temp. Diff. per Inch	Total Temp. Diff.	Temp. Diff. per Inch	Total Temp. Diff.	Temp. Diff. per Inch	Total Temp. Diff.	Temp. Diff. per Inch
Jan. 25, 1924												
9.30 A. M.....	10	5.0	21	7.6	*34	10.5	34	6.3	*14	2.7	48	4.5
9.50 A. M.....	11	5.5	*18	6.5	34	10.5	32	6.0	15	2.9	*47	4.4
2.56 P. M.....	*5	2.5	20	7.3	52	16.0	35	6.5	21	4.0	56	5.3
3.15 P. M.....	7	3.5	19	6.9	52	16.0	34	6.3	22	4.2	56	5.3
4.40 P. M.....	7	3.5	21	7.6	**62	19.0	**38	7.1	24	4.6	62	5.8
Average.....	8	4.0	20	7.3	47	14.5	34.5	6.4	19	3.6	54	5.1
Jan. 28, 1924												
2.20 P. M.....	16	8.0	28	10.2	49	15.0	32	6.0	16	3.0	48	4.5
3.46 P. M.....	16	8.0	40	14.5	54	16.6	*30	5.6	19	3.6	49	4.6
Average.....	16	8.0	34	12.4	51.5	15.8	31	5.8	17.5	3.3	48.5	4.6
Feb. 26, 1924												
1.30 P. M.....	6	3.0	21	7.6	54	16.6	30	5.6	22	4.2	52	4.9
3.15 P. M.....	12	6.0	36	13.1	54	16.6	32	6.0	24	4.6	56	5.3
Average.....	9	4.5	28.5	10.4	54	16.6	31	5.8	23	4.4	54	5.1
March 19, 1924..	17	8.5	42	15.3	58	17.8	35	6.5	**29	5.5	**64	6.0
June 18, 1924....	*20	10.0	**44	16.0	51	15.7	32	6.0	28	5.3	60	5.6
Grand Average.	11.6	5.8	28.2	10.3	50.4	15.5	33.1	6.2	21.3	4.1	54.4	5.1
Maximum	20	10.0	44	16.0	62	19.0	38	7.1	29	5.5	64	6.0
Minimum......	5	2.5	18	6.5	34	10.5	30	5.6	14	2.7	47	4.4

All temperatures in degrees Fahrenheit.
** Denotes maximum of all readings at a given level.
* Denotes minimum of all readings at a given level.

The temperature difference from B-5 to B-6 is omitted from this tabulation since the readings between these points show an increase instead of decrease in temperature from B-5 to B-6.

This tabulation, together with the previously tabulated readings, shows as might be expected that in general as the chimney shell becomes thinner, the actual temperatures in the concrete near the outside face become greater but that the *difference* in temperature between the inner and outer surfaces of the chimney shell becomes less. The readings also appear to show that as the chimney shell becomes thinner the slope of the temperature gradient becomes flatter whereas for practically unchanged temperature conditions, the slope of the true gradient must be steeper as the chimney shell becomes thinner. This discrepancy may be due to the small radial distance between the points where the readings were taken, so that a slight inaccuracy in the observed temperatures would materially affect the slope of the temperature gradient.

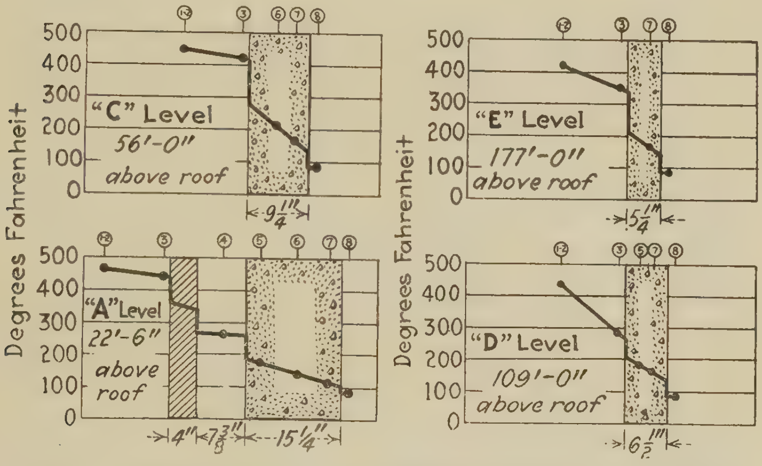


FIG. 5.—PROBABLE CHARACTER OF TEMPERATURE GRADIENT THROUGH CHIMNEY SHELL.

Tests of such limited nature cannot be taken as at all conclusive and there are many unexplained variations in the observed readings. However, until further experimental data on this subject are available, it is felt that for the purpose of designing reinforced-concrete chimneys subject to conditions similar to those existing for the chimney under discussion, the general trend of the test data warrants the conclusion that the maximum drop in temperature through the concrete of the chimney shell may be taken as:

(a) 20 deg. F. per inch of shell thickness where the chimney is unlined.

(b) 10 deg. F. per inch of shell thickness where the chimney is lined so as to furnish insulation equivalent to that of the lining in the chimney tested.

It is hoped that these tests will lead to further tests along the same line, and will result eventually in an accumulation of data from which still more positive conclusions can be drawn.

DISCUSSION.

J. W. LOWELL.—This is just the kind of data we need upon which to base a theory of the design of concrete chimneys. This is the first real data we have had on the difference in temperature between the inside and outside of concrete chimney shells. Mr. Lowell.

PROF. IRA H. WOOLSON.—What was the average high temperature registered in this chimney and for what periods of time did that high temperature exist? Also, was the interior of the chimney examined since the tests were conducted? Was any influence noticeable upon the concrete, and if so, what? Prof. Woolson.

MR. DOCKSTADER.—The temperatures varied at every point in the chimney shell, and also varied somewhat dependent on the outside temperature. The amount of heat radiated through the chimney shell affects the temperature of the gas inside the chimney shell. The maximum recorded temperature of the gas on the inside, as I recall it, was something near 450 deg. near the bottom of the chimney, and varied down to near 400 deg. near the top. The chimney has not been examined since the test was made. The chimney is designed to take care of three 1608-h.p. boilers operating at 250 per cent rating. Mr. Dockstader.

PROF. M. O. FULLER.—What was the length of time taken to determine the coefficient of expansion of that concrete so that the strains in the chimney could be investigated? Prof. Fuller.

MR. DOCKSTADER.—I have been trying to find out if any experiments have been made to determine the coefficient of expansion of concrete. I do not believe that very much has been done along that line. With the extreme variation in the rest of the computations, a slight difference in that coefficient would not make much difference, but I think it is a subject that might be investigated along the same lines that the coefficient of expansion of gunite were very accurately determined. Mr. Dockstader.

Inundation as a Practical Aid to Uniform Concrete.

Arthur A. Levison.*

The last few years have brought a keen realization that concrete as made for construction purposes is notably lacking in uniformity of quality. Over 120,000,000 of cubic yards of concrete were placed during 1924. It is estimated that the consumption of cement was sufficient to pave a concrete road 18 ft. in width and of a length equivalent to 20½ times the Lincoln Highway, extending from coast to coast. With the yearly production of concrete reaching so tremendous a total, the quality of the concrete produced assumes an important aspect. The lack of uniformity cannot be attributed to any lack of ability or general laxity on the part of those connected with the design, supervision, or manufacture of concrete mixtures. In fact, there has been more recently a concerted and energetic effort to improve the quality and the uniformity of that quality which has been attended by some measure of success.

Standard adequate methods of tests have been developed as regards the materials commonly employed for making concrete. By means of these tests, we are enabled to select with a great degree of assurance the water, the sand, the stone or gravel, and the cement which are required for the mix. We can at least ascertain the quality of the materials available and form definite judgment as to their concrete-making properties. There have been developed ingenious formulas or methods for combining these materials in such a way as to provide a definite quality of the resultant concrete. The designing of concrete mixes for various materials and for various purposes has reached a stage of perfection that leaves little to be desired for the present at least. Why then the deficiency of quality and uniformity in the concrete? To the close observer of construction methods as ordinarily practiced the answer to this question is quite apparent. While the theory of concrete and concrete materials has been subject to diligent study and research, we have failed to give the proper attention to the field methods employed on the job. We have neglected to give sufficient consideration to the practical construction processes commonly employed, to the equipment ordinarily used and its limitations, and to the field control necessary to make a perfect product.

Non-Uniformity Investigations.—In the spring of 1924 the United States Bureau of Public Roads, being aware of the non-uniformity of the concrete entering into highway work in the various states where it administers the expenditure of federal-aid funds, began the collection of data

*Chief Engineer, Road Department Blaw-Knox Co., Pittsburgh, Pa.

which would indicate just how much uniformity was being obtained. Table No. 1 illustrates data which are typical of that furnished by a number of representative states. The states included in this tabulation employ what are considered to be advanced methods of construction. They have rigid specifications, excellent laboratories for testing and research, and are noted for their high quality of construction. Yet the quality of the concrete, as evidenced by the ranges of strengths of cores drilled from individual pavements, demonstrated a surprising lack of uniformity; the maximum strength being often twice the minimum and more. Another state highway department which did not furnish the results of its tests in actual strengths, reported an extreme range of strengths from 34 to 174

TABLE I.—DATA ON NON-UNIFORMITY OF CONCRETE STRENGTH AS REVEALED BY DRILLED CORES: FROM 27 STATE HIGHWAY DEPARTMENTS.

State	A			B			
Project	1	2	3	1	2	3	4
Mix	1:2:4	1:2:3½	1:2:3½	1:2:3½	1:2:3½	1:2:3½	1:2:3½
Average Strength	3200	2480	3950	4080	3910	4340	3960
Minimum Strength	2340	1800	2660	3660	2840	3060	3030
Maximum Strength	4300	3360	5370	5050	5140	6420	5640

State.	C				D			
Project	1	2	3	4	1	2	3	4
Mix	1:2:3½	1:2:3½	1:2:3½	1:2:3½	1:2:3	1:2:3	1:2:3	1:2:3
Average Strength	4860	4520	4380	3900	4260	5220	4810	3900
Minimum Strength	4000	2380	3020	2460	3460	4140	3980	3020
Maximum Strength	5680	6520	6570	5870	5140	5850	5980	5020

per cent of the average strength, this occurring with drilled cores five months old at the time of test. The non-uniformity of concrete as illustrated by these data is all the more striking when it is taken into consideration that on all of this concrete paving construction the highest type of field control prevailed, specifications were exact and more rigidly enforced, and every effort was put forth to make good, honest concrete for lasting service.

The United States Bureau of Public Roads then circularized every state highway department in the country in an effort to secure an expression of opinion as to the reasons for the non-conformity of concrete. Table No. 2 represents the consensus of opinion of the twenty-seven highway departments who had sufficient experience with concrete paving work to venture an opinion based on that experience. From this table it will be seen at a glance that by far the greater number of suggested causes for

TABLE 2.—SUGGESTED CAUSES FOR NON-UNIFORMITY OF CONCRETE: STATE HIGHWAY DEPARTMENTS.

1. Cement	
(a) Variation in cement from one plant	2
(b) Variation between different brands of cement	3
(c) Storage of cement	1
2. Water	
(a) Variation in quality	2
(b) Acidity or alkalinity	2
3. Aggregates	
(a) Variation in cleanliness	2
(b) Variation in quality	2
(c) Variation in grading	7
4. Construction Processes	
(a) Variation in consistency of concrete	21
(b) Inaccuracy of measuring aggregates	12
(c) Segregation of coarse aggregates in stock piles	2
(d) Bulking of sand due to moisture	5
(e) Segregation of concrete in mixer and in pavement	5
(f) Variation in time of mixing	9
(g) Oversanding	1
(h) Arbitrary proportions	2
(i) Insufficient tamping	5
(j) Too much tamping	1
(k) Use of boom and bucket on mixer	1
(l) Variable cement factor	1
(m) Equipment in poor condition	1
(n) Variation in manipulation of concrete	4
(o) Variation in thickness of pavement	1
5. Curing	
(a) Variation due to wet and dry subgrades	1
(b) One section shaded, another exposes to sun	1
(c) Non-uniform curing	6
(d) Variation in nature of material used for cover	1
(e) Variable absorption of subgrade	2
(f) Inadequate curing	5
(g) Absorption by dry subgrade	1
(h) Temperature changes during construction period	3
(i) Rise or fall of temperature during setting period	1
(j) Effect of freezing	2

non-uniformity fall under the heading of construction processes. In fact, the suggestions under this heading are almost twice as numerous as those under all the other headings combined. It is pertinent to note particularly that twenty-one states pointed to the valuable consistency of concrete as a contributing cause of non-uniformity. No other single defect received such unanimous acclaim.

Thoughts on Concrete Making.—To overcome the undesirable results previously referred to, we must for the time at least turn from the thought on the theory of concrete to the more practical aspects of its manufacture. We should guide our research in such a manner as will result in improved methods and equipment for construction. The methods and equipment have never been developed to such a point as to counteract adequately the disconcerting variables and conditions encountered on the job. We have been making concrete in a somewhat haphazard manner and have required a large factor of safety to insure safety of the structure. Even so, failures have been altogether too numerous. This sort of practice is not only undesirable, but utterly lacking in economy.

The manufacture of concrete consists of the mixing together of the necessary suitable ingredients in their proper proportions. The concrete is then transported and placed in the structure and allowed to harden with protection against rapid drying out and extremes of weather and temperature. Stated in this manner, the proposition does not seem to involve any great complexities, yet there are conditions which introduce difficulties in accomplishing efficiently what appears to be so simple a process.

Not many years back the common practice in measuring the aggregates for the batch was by wheelbarrows or similar conveyances. Aggregates were stored on the ground and became mixed with dirt. These conditions still prevail in some localities, but they are being rapidly replaced either by measuring boxes, which are filled from overhead storage bins and struck off by hand, or by automatic measuring devices which are also filled from overhead bins and are automatically struck off to a constant level. These advanced schemes of measuring aggregate provide more constantly accurate measurements of the materials for each batch and permit cleaner aggregates to enter the work. In addition to this advantage, it has been found from experience that automatic measuring devices also provide for time and labor-saving on the job. For this reason equipment of this kind has found its way into construction work where it is not required by the specifications for the work. Together with these measuring devices, a calibrated tank attached to the mixer or independent of it, delivers a predetermined amount of water to each batch of concrete. Yet even with these refinements of construction, there are important variables which are now fully recognized and which have a direct effect on the strength and quality of the concrete.

Effects of Sand Bulking.—The variable bulking or swelling of sand, due to a fluctuating moisture content upsets the apparently correct volu-

metric measurements of the fine aggregate to a remarkable degree. It is possible to make volumetric adjustment for the bulking (as is being done on some work), but the degree of accuracy and uniformity of the measurements would depend entirely on the close inspection and tests of the fine aggregate as it was being used. It would be necessary to vary the adjustment for bulking with the variable bulking. This may involve the necessity of making numerous troublesome changes in the volume of fine aggregate and in the measuring equipment. The construction year just past has seen a large amount of attention given to the measurement of fine aggregates with correction for bulking. A few examples of agencies who have only recently taken steps to compensate for the bulking of sand on concrete construction are the Missouri Highway Department, the California Highway Department, the Big Four R. R., the Pennsylvania R. R.

The concrete bridge built at Becks Run, by the Pennsylvania R. R., is a notable example of what can be done with proper control of the construction processes. On this job automatic volumetric measuring devices were used, the bulking of sand was corrected, and the mix was proportioned in accordance with the fineness modulus scheme. As a result of these improved field methods it was found possible to produce, approximately, concrete of predetermined strength, with a reduced variation in the quality.

In 1923 the Iowa Highway Department built two concrete paving projects measuring the aggregates by weight. As a result of this work, weight measurements were adopted as standard practice for 1924. The chief advantage to be gained from the weighing method is in the measurement of the fine aggregate for the mix. Bulking is automatically corrected, the sand being weighed out as so many pounds for each batch, with a proper correction for the weight of contained moisture. No other major advantage can be claimed for weighing. But today we find the practice of weighing has spread to other branches of general concrete construction, to some extent, and a number of highway departments are seriously considering abandoning volumetric measurement for weighing.

Control of Water.—But there is one striking variable which has a far-reaching effect on the quality, strength, workability, and economy of concrete. That is, the measurement of the water for the batch. Even assuming that a constant amount of water is added to each batch of materials, the water-cement ratio would not necessarily be constant. The reason for this is the variable moisture contained in the aggregates, particularly the fine aggregate.

On a paving job in Pennsylvania, with which the writer had personal contact, the amount of water contained in successive batches of sand varied from 3 to 10 per cent by weight. With a six-sack batch of a 1 : 2 : 3 mix, the water content of the individual batches of sand in a four-compartment truck would vary from 30 lb. to 95 lb. This means that a variation of more than a cubic foot of water per batch actually existed as the batches came to the mixer.

Even a good mixer operator is helpless as far as making an accurate correction for this variation is concerned. More often no correction of any kind is made, the same amount of water being added to successive batches and a variation of almost two-tenths in the water-cement ratio results. Under such conditions weighing of the aggregates would be of practically no value in connection with the control of the water for the mix.

Inundation.—A construction process of recent development accomplishes for the art of making concrete that which is most desirable as regards the accurate and uniform measurement of the fine aggregate and water. This process is termed "inundation"—and it consists of measuring the sand and water for the mix simultaneously and together in the one container or measuring device.

It has been demonstrated beyond doubt that when sand is measured in an immersed or inundated condition, the bulking action is automatically destroyed and the volume of the inundated sand is closely comparable to the volume of the same amount of sand in a dry condition. Further, the amount of water required to inundate a constant volume of a given sand always remains constant. This is true regardless of whether the original condition of the sand prior to inundation was dry, moist, or saturated. It can be readily seen that with inundation properly adapted to concrete construction, two distinct advantages accrue; first, the constantly uniform measurement of the fine aggregate and, secondly, accurately uniform measurement of the water for each batch. Thus two of the most troublesome constructional difficulties are solved with one stroke.

It is thought by many engineers that the volumetric measurement of sand in a loose, bulked condition is on the side of safety, because it results in a richer mortar and, therefore, a stronger concrete. This theory is not supported by the results of tests conducted by the United States Bureau of Public Roads in 1924 on sample batches of concrete made up with sand measured dry, and moist sand measured loose. These test results indicated definitely that for ordinary mixes the concrete made with the sand measured in a dry condition was equally as strong, and in most cases stronger than the concrete made with bulked sand measured loose. Both gravel and broken stone concrete were used in these tests. Further, the difference in yield was approximately 8 per cent with the advantage accruing to the dry sand batches. Accordingly, with sand measured in an inundated condition there is a resultant economy with no loss of strength. This is explained on the basis that the concrete made with the sand measured in a compacted condition was denser and the increased density more than made up for the diminution in richness of the mortar.

The equipment for the adoption of the inundation method to construction has already been the subject of considerable study and experimentation. Construction equipment to be successful must not only accomplish adequately the purpose for which it was reduced, but it must be rapid acting, labor saving, accurate, and durable. These requirements have placed a heavy burden on those engaged in developing the equipment re-

quired for applying the inundation method to the art of making concrete.

Following are sketches showing various developments of the inundator, together with an explanation of their operation:

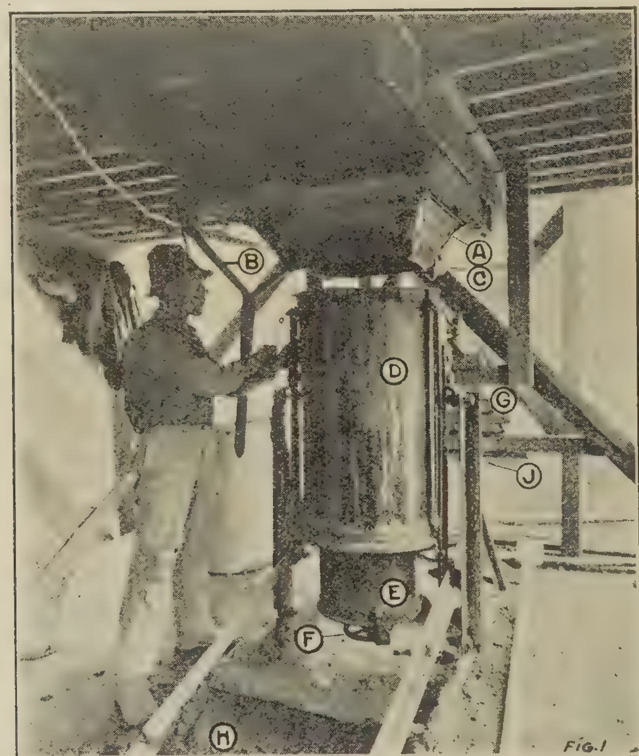


FIG. 1.—INUNDATOR USED IN AND NEAR NEW YORK.

This type of inundator has been used successfully on the construction of a large number of reinforced-concrete buildings in and near New York City, by the White Construction Co., who are the inventors of the process. The inundation method of obtaining accurately the correct measurement of sand and water for the mix, together with the devices for accomplishing this are all covered by letters patent. "A" is a steel neck attached below the opening in the bin bottom of the sand storage. "B" is a lever which operates a shaker gate in the neck "A". "C" is the water supply line. "D" is the inundator proper, which consists of a cylindrical steel container, with an overflow spout near the top, and a sand trap separating it from "E", the chamber in which the excess water for each batch is measured. "F" is a hand wheel which moves the bottom of chamber "E" either up or down to adjust the water measurement. In operation, the operator turns a valve and fills the inundator with water to an arbitrary level, about half full. By working the lever "B" back and forth, the sand streams into the inundator from the bin until the container is full, the excess water overflowing from the spout and being carried off by the flume "J". A latch is released and the inundator dumps, turning on the pivot "C" and discharging sand and water into the mixer hopper "H".

FIG. 2.—ANOTHER VIEW OF INUNDATOR.

Fig. 2 is another view of the proportioning plant, showing the other side of the inundator. The overflow spout is shown feeding into a wooden flume. This overflow spout is adjustable vertically on the inundator to vary its capacity. The batcher shown in the background was used for measuring the coarse aggregate volumetrically in an automatic manner. This plant, while applying the principle of inundation, does not provide the complete accuracy and automatic operation required for close control. It is possible for the operator, through carelessness, to over-charge the inundator. If the size of the batch were to be reduced, the overflow spout would be moved downward and an interior level used for measuring the volume of inundated sand. This is quite undesirable.

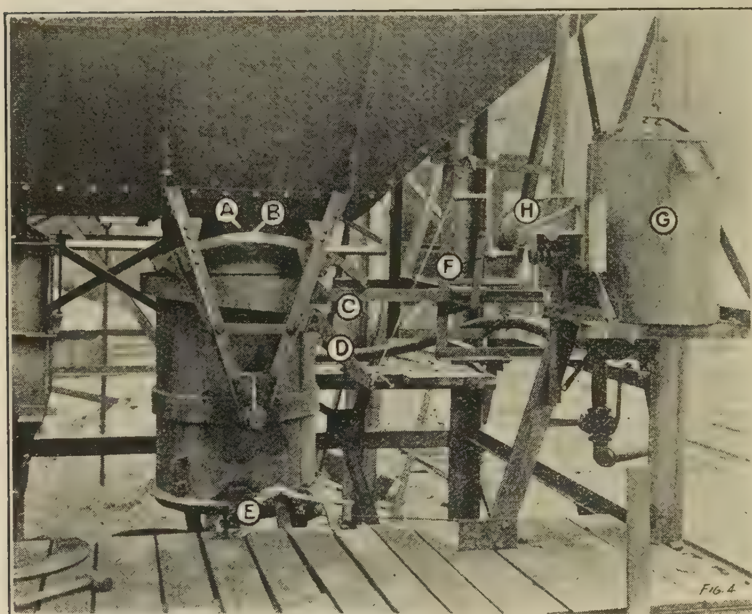


FIG. 3.—INUNDATOR SUSPENDED FROM BIN.

In this view the inundator is shown suspended beneath the bottom of a steel storage bin and the adjustable auxiliary water measuring tank is shown in detail "G". In this installation the adjustment of the auxiliary water measuring device is a winch shown at "H", which is operated by means of a handle.

The problem of adapting inundation to other than central plant mixed concrete has not as yet been completely solved. It will be realized that for highway work where the paver is on the road, practical difficulties are encountered in connection with the hauling of the inundated sand and the water for the mix. In addition, if the wet materials are deposited in the ordinary mixer skip and the cement is then added, it would involve considerable extra labor to make the skip discharge cleanly into the mixer drum. It may be that the inundated sand and water can be hauled out to the mixer in watertight batch boxes. Substituting a batch hopper for the usual skip loader, a crane could hoist and dump the batch into the mixer hopper. The other alternative for employing the inundation method would be to construct pavements from a central mixing plant. The first weighing job in Iowa was a paving project some twenty miles in length, which was successfully built from central mixing plants.

Inundation is the most logical and advantageous step of recent years to help in securing that most desired uniform quality of concrete. Uniform concrete means safe, dependable concrete. It means also better service from the structure and greater economy of construction. We can profitably turn our attention to the consideration and devising of ways and means to correct the faults atendant on concrete construction as practiced today. It would be of invaluable benefit if research were directed more along these channels for the development of superior schemes of controlling construction, and the investigation of better methods and equipment for concrete construction.

DISCUSSION.

Mr. Jackson.

F. H. Jackson (*By Letter*).—The author cites a survey made by the U. S. Bureau of Public Roads for the purpose of ascertaining the causes of non-uniformity of concrete in pavements as revealed by tests on drilled cores. He shows that of all the possible causes for non-uniformity which were advanced by 27 state highway departments, variations in construction process were by far the most numerous, and that among these the outstanding cause was variable consistency. I believe that variable consistency is the principal cause of non-uniformity. All recent investigators recognize the importance of this factor in controlling the strength of concrete, so much so that we read that "one pint of water in a one bag batch over the amount required wastes as much as two or three pounds of cement." The principle that the smallest amount of water consistent with the proper placing and finishing of the concrete should always be used has become almost axiomatic among engineers. So much for theory.

When we come to the practical application of this theory, however, we find that, although great progress has been made within the last few years, there is still much room for improvement. One has only to remain a single day on the average concrete paving job to realize that the consistency of the concrete is far from uniform, and furthermore that under present methods of operation, it is not a simple matter either to make or keep it so. Let us consider a six-bag batch of 1 : 2 : 4 concrete. We will say that 12 cu. ft. of sand, as measured, weigh 1,000 lb., and contain 4 per cent of moisture, a normal stockpile condition. We will also assume that under these conditions the mixer operator must add 260 lb. (31 gal.) of water to obtain the desired consistency. The sand already contained 40 lb. of water, making 300 lb., or 36 gal., the total water content. Suppose, now, the moisture content of the sand was increased by as little as 2 per cent, other factors remaining the same. We automatically add 20 lb., or 2½ gal. of water to our batch, thereby changing the water-cement ratio from approximately 0.8 to about 0.85. This may seem a very small change, but when we consider that it was brought about by a moisture change in the sand so slight as to be unnoticeable to the eye, we begin to realize some of the difficulties which the engineer on the job is up against in his efforts to maintain a uniform consistency. Anything which tends to cause a change in the amount of water required for a given consistency will, of course, change the amount of water which must be added to the batch. For instance, it is surprising how comparatively slight changes in the gradation of the coarse aggregate will affect the workability of the mix, and consequently the amount of water which the mixer operator must either add or leave out in order to keep his consistency constant.

TABLE I.—MECHANICAL ANALYSIS USED IN EXPERIMENTS.

Per Cent Between.	Sand Number. A	B	C	D
$\frac{1}{2}$ in. to $\frac{1}{4}$ in.	0	0	15	0
$\frac{1}{4}$ in. to No. 10	16	43	25	0
No 10 to No. 20	20	32	29	12
No. 20 to No. 30	24	7	10	32
No. 30 to No. 40	9	3	5	13
No. 40 to No. 50	13	5	7	11
Under No. 50	18	10	9	32

TABLE II.—INUNDATION TESTS OF SAND=1:2:4 MIX

Sand.	A	B	C	D
Wt. dry sand (2500 cc.), grams	4060	4265	4285	3840
Wt. sand when inundated, grams	4980	5125	5150	4925
Wt. water required to inundate, grams	920	860	865	1085
Flow (inundation water only)	102	102	104	106
Wt. water required for 160 flow, grams	280	325	320	300
Total water for 160 flow, grams	1200	1185	1185	1385
Water-cement ratio	.96	.95	.95	1.11
Wt. damp sand (2500 cc.), grams	3295	3330	3315	2725
Wt. add. damp sand to fill inundator, grams	1025	1135	1260	1400
Total weight damp sand, grams	4320	4465	4575	4125
Actual weight sand, grams	4075	4210	4315	3890
Weight damp sand when inundated, grams	5020	5105	5145	4950
Weight water required to inundate:				
Original water in sand (6 per cent), grams	247	255	260	235
Additional water required, grams	700	640	570	825
Total, grams	947	895	830	1060
Flow (inundation water only)	105	104	102	108
Wt. additional water required for 160 flow, grams	260	300	340	300
Total water for 160 flow, grams	1207	1195	1170	1360
Water-cement ratio	.97	.96	.94	1.09

TABLE III.—INUNDATION TESTS OF SAND=1:3:6 MIX

Sand.	A	B	C	D
Wt. dry sand (2500 cc.), grams	4060	4220	4275	3825
Wt. sand when inundated, grams	5000	5120	5140	4880
Wt. water required to inundate, grams	940	900	865	1055
Flow (inundation water only)	108	125	118	108
Wt. water required for 160 flow, grams	260	240	265	240
Total water required for 160 flow, grams	1200	1140	1130	1295
Water-cement ratio	.96	.91	.90	1.04
Wt. damp sand, grams	3250	3330	3315	....
Wt. damp sand required to fill inundator, grams	1080	1240	1255	....
Total wt. damp sand, grams	4330	4570	4570	....
Actual weight sand, grams	4090	4310	4315	....
Weight damp sand when inundated, grams	5040	5160	5180	....
Wt. water required to inundate:				
Original water in sand (6 per cent), grams	240	260	255	....
Additional water required, grams	710	590	610	....
Total, grams	950	850	865	....
Flow (inundation water only)	110	112	120	....
Wt. additional water for 190 flow, grams	240	220	260	....
Total water for 160 flow, grams	1190	1070	1125	....
Water-cement ratio	.95	.86	.90	....

Many laboratory investigators have shown that changes in gradation of coarse aggregate do not affect the strength to any great extent, as long as the voids are filled. This may be true under laboratory conditions where complete control is exercised over the amount of water used in the mix. No such rigid control over the water is possible in the field, however, under present operating conditions. The result is that, although changes in gradation of coarse aggregate may not directly affect the strength of the concrete, they markedly affect the workability of the mix, and thus indirectly affect the strength. It is my opinion that variations

TABLE IV.—COMPRESSIVE STRENGTH—7 DAYS: 6-IN. BY 12-IN. CYLINDERS.

Sand.	1:2:4 Mix.		1:3:6 Mix.	
	Wet.	Dry.	Wet.	Dry.
A—Well Graded	840 728 740	906 763 730	366 300 375	330 371 275
	769	800	347	325
B—Coarse	650 591 588	800 820 778	339 369 ...	388 370 ...
	610	799	354	379
C—15 per cent Oversize	882 793 812	896 812 1067	398 383 373	345 314 278
	829	925	375	312
D—Fine	631 776 ...	849 835 796	335 376 307	305 309 267
	703	827	339	294

in moisture content of sand and gradation of coarse aggregate affect the quality of the concrete largely because of their effect on the water requirements of the batch, and that until we have provided some means of securing uniformity in both items, it is impossible to have absolutely uniform concrete.

Having in mind the possible future application of the inundation method of measuring sand to road construction, the Bureau of Public Roads recently conducted a short series of tests to ascertain to what extent the amount of water required to inundate the sand took care of the water requirements of the concrete in which the sand is to be used. Sands of four different gradings were employed in the experiment, as well as concrete of both 1 : 2 : 4 and 1 : 3 : 6 mix by volume. Well graded gravel between ¼ and 1½ in. in size was used for both mixes. The sands varied from extremely coarse to very fine, and the gradings are shown in Table 1. Two sets of specimens were made: one in which the sands were inundated

when in an air dry condition, and the other in which the sands contained 6 per cent moisture previous to inundation. Table 2 shows the relative amounts of water required to inundate each of the four sands both in the dry and damp condition, as well as the amount of additional water required in each case to produce a flow of 160. It will be observed that in all cases that the amount of water to be added to produce a consistency of 160 flow was approximately the same, regardless of the initial moisture condition of the sand. This fact, as has been pointed out, is of interest in view of the influence which the moisture content of the sand has upon the water requirement of the concrete when proportioned in the ordinary manner. It will also be noted that the amount of water required to inundate the sand is always considerably less than the total amount required to produce a workable consistency. This holds true for all four gradings of sand and for both mixes. This fact is also important in connection with a study of the inundation method because if it were not true, it would be necessary to withdraw water from an inundated sand before mixing the concrete. These laboratory investigations were of course made on a small scale, using only small quantities of materials. It would be of interest if figures were available showing just what degree of accuracy was possible under field conditions.

As the author states, the practical application of the inundation method to paving work is not apparent at this time, except insofar as central plant mixed jobs are concerned. Ways and means for surmounting the practical difficulties involved will, however, probably be worked out in the near future. In this connection it should be borne in mind that inundation is not the only method which may be used for correcting the bulking action of damp sand. The same result may be accomplished by weighing the sand as is done in Iowa on paving work, or by simply correcting the volume of the measuring device to compensate for the bulking action. The inundation scheme is, however, as far as I know, the only method whereby the effect of variations in moisture content of the sand are cared for automatically.

In conclusion, I wish to second Mr. Levison's plea for a concentration of effort along the lines of practical field control of construction process. Let us learn not only how to design and make uniform concrete in the laboratory but also how to make it more uniform in the field. Much progress along this line has been made within the past year or two in the general construction field. Why should not similar investigations of improved methods of field control be initiated in the highway field? It is only by so doing that we will reap the full benefit of the splendid research work which has been done.

DISCUSSION.

Mr. Crum.

R. W. Crum (*By Letter*).—We are much indebted to Mr. Levison for his clear presentation of the necessity for improvement in construction methods, and description of the apparatus used for the inundation process.

It is unquestionably true that our knowledge of the science of concrete mixtures is far ahead of the art of actually making the concrete, and a most important duty right now is to develop methods for taking full advantage of our scientific knowledge. The first step that must be taken is to provide for accurate and uniform measurement of the ingredients for the concrete, and this we now realize means the water as well as the cement and aggregates. Accurate measurement of the aggregates is a simple problem easily solved. They can be weighed or the sand can be measured in an inundated condition as described by Mr. Levison. Improvement in water measuring devices is sorely needed. Last summer some studies were made on two paving jobs in Iowa, of the water-cement ratio of the concrete as actually laid. The aggregates were weighed and a reasonably careful attempt was made to keep a uniform consistency, yet in 38 determinations the water-cement ratio varied from 0.684 to 1.072 on one job and from 0.648 to 0.959 on the other. This range in water-cement ratio is enough to cause a variation in strength of 100 per cent. Some new device for measuring water must be developed to cure this condition.

The "inundation method" offers such a device for central mixing plants for there is no question but that sand and water can be accurately measured together in the same container. Whatever device is used for measuring the water must be readily adjustable to allow for variations in the moisture content of the sand. I feel sure that with the general realization of the necessity and demand for accurate measuring apparatus that our equipment men will produce the needed machinery.

Mr. Ahlers.

Mr. Ahlers.—Is this inundation method applicable to premixed aggregates?

Mr. Levison.

Mr. Levison.—I would say that the subject has not been investigated to such an extent that an answer to that question can be given definitely.

Mr. Egelhoff.

R. F. Egelhoff.—The inundation method as described no doubt is a great advance in the production of good concrete on the job, still there are some items unexplained in the matter of water control. One of these is the water content in the gravel. We know that after a rain our gravel contains a great deal of water, both through absorption and outside coating. This would be something worth while investigating with an idea to devising some way of measuring that water content, thereby getting a closer control on the water.

DISCUSSION.

Prof. Slater.—The question was asked whether the inundation method would apply to premixed aggregates. I cannot answer the question finally, but I suspect it would not, because you would come up against the case where it took more water to inundate the total aggregate than required in the concrete, and you would have to get some of the water out of it after you inundated it. Prof. Slater.

Mr. Levison.—Inundation was used on the Mare Island Navy Yard work under conditions where more water was required for inundation than was required for the consistency of the concrete. The process resorted to was drawing off some of the water after the batch was inundated and measuring the water drawn off as a fraction of that required for inundation. Then they went ahead with the batch. Mr. Levison.

Proportioning Concrete in Products Plants.*

By Stanton Walker.†

Introduction.—The manufacturer of concrete products has unusually favorable opportunities for controlling the quality of his product with precision. He has a permanent plant, protected from the elements. Ample space is generally available for storage of aggregate and cement. He is able to choose from available materials, and by reason of the steady market which he establishes, he can demand the kind of materials needed. Frequently he produces his own aggregate, in which case its quality and uniformity is limited only by the quality of the supply and the attention which he is disposed to give to it.

The adoption of uniform standards for concrete products throughout the country, particularly for building block and tile, tends to place competition on a price basis rather than on a quality basis. This is as it should be. There is no advantage in making a 2000-lb. block if a 1000-lb. block meets all requirements, neither is it good business to make only 15 or 16 blocks per sack of cement if 20 or more equally good blocks can be made merely by the expenditure of a little effort toward proper selection and grading of aggregate.

The purpose of this paper is to point out methods by which products of the necessary quality can be uniformly produced at the least cost. These methods have been developed at the Structural Materials Research Laboratory, which during the past nine years has carried out many thousands of tests on the effect of grading and quality of aggregate, quantity of cement, quantity of mixing water and related factors as a part of its study of concrete and concrete materials. Similar investigations have been carried out at many other laboratories.

Water-Ratio Method of Proportioning Concrete.—The most far-reaching result of these researches was the development of the water-ratio theory which affords an accurate method for proportioning concrete. This method has for its basis the fact that the strength of concrete is fixed by the ratio of volume of mixing water to volume of cement in the concrete. The smaller the water ratio, the higher the strength.

It has long been common knowledge that for definite conditions of manipulation, curing, workability and age, the strength of concrete is fixed by the following three factors so long as the concrete is workable, and so long as clean and durable aggregates are used:

Size and grading of aggregate;
Quantity of cement;
Consistency of concrete.

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*This paper, presented before the 1925 convention of the American Concrete Institute, was accompanied by a demonstration of tests of aggregates.

†Associate Engineer, Structural Materials Research Laboratory, Lewis Institute, Chicago.

The development of the water-ratio theory showed that the use of coarser aggregates up to the limit of workability, the addition of more cement, and the use of drier concrete within the limit of plastic mixes increase the strength only because the quantity of mixing water required is reduced.

An almost indispensable corollary to the water-ratio theory is the definite relation between quantity of mixing water required for given conditions and the grading of the aggregate as measured by the fineness modulus.

Sieve No.	Size of Clear Opening, Inches
100	.0058
48	.0116
28	.0232
14	.046
8	.093
4	.185
⅜ in.	.375
¾ in.	.75
1½ in.	1.50

The important characteristic of these sieves is that each has a clear opening double that of the next smaller sieve.

The fineness modulus can be used to estimate the effect of changes in grading of aggregates on the strength of concrete, since it fixes the quantity of mixing water required. For a detailed discussion of water-ratio and fineness modulus, see Bulletin 1 of the Structural Materials Research Laboratory, "Design of Concrete Mixtures," by Duff A. Abrams.

Tests of Concrete Products.—Most of the researches from which these fundamental laws were developed were carried out on plastic mixtures. During the past three years, however, a number of series of tests on machine-made block, brick and tile have been carried out in concrete products plants. These investigations included tests of the effect of size and grading of aggregate, quantity of cement, plasticity of concrete, type of aggregate, curing condition, time of mixing and age at test. They showed that the strengths of machine-made products follow with minor variations, the laws developed from tests of plastic concrete except where the *consistency* of the concrete is involved. For plastic mixtures the strength increases as the concrete becomes drier, up to the limit of workability; beyond this point the strength drops off owing to the difficulty of placing the concrete in a compact mass. In machine-made products it is necessary to use a mixture dry enough to prevent the unit from sagging on removal from the machine. Under this requirement the maximum quantity of mixing water which can be used is approximately equal to, or slightly less than the amount which produces maximum strength. Consequently in concrete products manufacture the effort should be to use as wet a consistency as conditions permit.

The following brief statements summarizing the results of tests of building units will be of interest:

(1) Mixtures containing coarse aggregate gave strengths in concrete building units as much as 75 per cent higher than those made with sand alone. Gradings of aggregate which gave the highest strength for a given quantity of cement in general, produced block having a rough surface texture.

(2) Increasing the cement content increased the strength in the proportion of about 1 to 2 per cent for each 1 per cent of cement added. A 1:4 mix (about 12 8x8x16-in. blocks per sack of cement) will, under average conditions, produce two to three times the strength obtained from a 1:8 mix (about 20 8x8x16-in. blocks per sack).

(3) The use of as wet a consistency as possible without causing the blocks to slump appreciably on removal from the machine gave in certain tests 25 to 50 per cent higher strengths than for the extremely dry mixtures. This was undoubtedly due to the fact that the wetter mixtures could be compacted better than the dry ones.

(4) The type of aggregate, so long as it was clean and structurally sound, had little effect on the quality of the product except insofar as it affected the workability and quantity of water required. Considerable amounts of dust in aggregates may in some cases improve the quality of the product due to the better workability obtained.

(5) The most favorable condition for curing was in a moist atmosphere of reasonably high temperature (100 deg. F. or more). It appears from these tests that the introduction of dry steam or high pressure steam into a curing room should be avoided.

(6) Mixing the concrete 2 to 3 minutes increased the strength of products about 35 per cent over the strength produced by 1 minute mixing. An earlier investigation of plastic concrete mixtures showed similar results.

(7) Products of average mixes cured in moist steam 24 hours and the remainder of the time until test in air, showed the strength at 7 days to be about 70 per cent of the 28-day strength. The ratio of the strengths at different ages varied with the quality of the concrete.

*Selection of Proportions for Concrete Products.**—The chief purpose of this paper is to describe methods of testing and proportioning aggregates to make the best use of available materials. The selection of proportions for *plastic mixtures* is greatly simplified by the fact that the method of placing the concrete has relatively little effect on its strength. Consequently, the proportions can be selected with a fair degree of accuracy from the average results of tests carried out in a laboratory. In the case of machine-made products, however, the method of manipulating the concrete plays such an important part in the strength of the product that, in the light of information at present available, it is desirable for each manu-

*For a more comprehensive discussion of this subject, see "The Manufacture of Concrete Masonry Units," published by the Portland Cement Association.

facturer to carry out tests over a wide enough range in mixtures and gradings of available materials to establish the proportions which will produce the desired strength at least cost.

The proportions for these tests should be expressed in terms of volume of mixed aggregate measured under standardized conditions (dry and rodded) an fineness modulus. They can then be reproduced readily, even though the grading of the available aggregates should change.

Suppose, for example, that experiments show that one sack of cement, 5 cu. ft. of sand and 3 cu. ft. of pebbles (aggregates measured damp and loose) are required to produce the quality of unit desired with the aggregate available. The first step toward placing this information in the desired form is to reduce the given volumes of aggregate to standard conditions of measurement. This is necessary because the amount of actual aggregate contained in a given volume measured damp and loose may vary as much as 25 per cent due to the presence of moisture and method of measurement.

A simple method of determining the difference in volume of materials measured damp and loose under plant conditions and the volume measured under standard conditions is as follows:

Fill a convenient measure having vertical sides (preferably cylindrical in form) with the aggregate in such a way that job conditions are duplicated as nearly as practicable, strike off, and weigh. Dry and weigh the entire sample. Replace the dried sample in the vessel in three layers, rodding each layer 25 to 30 times with a 5/8-in. round rod pointed at the lower end. Level and measure the depth of aggregate in the vessel. (This method of placing the aggregates in the moisture conforms to the method recommended by the American Society for Testing Materials for determination of unit weights.) This percentage of bulking and moisture content may now be calculated. Suppose that a vessel 11 in. deep is used, that it holds 18 lb. of moist sand, and that after drying the sand weighs 17 lb. and fills the vessel to a depth of 8 3/4 in.

$$\text{The moisture content is } \frac{18-17}{17} = \frac{1}{17} = 0.059 = 5.9 \text{ per cent.}$$

$$\text{The bulking of the sand is } \frac{11-8.75}{8.75} = \frac{2.25}{8.75} = 0.26 = 26 \text{ per cent.}$$

For this percentage of bulking the 5 volumes of sand measured damp and loose as assumed in the problem, represent only 4 cu. ft. of dry and rodded sand.

$$\left(\frac{5}{1.26} = 4.\right)$$

Assume that the bulking of the coarse aggregate due to moisture and method of measurement determined by the same method was found to be

10 per cent. Then the 3 cu. ft. of pebbles occupy 2.7 cu. ft. measured dry and rodded. The mix, therefore, expressed in terms of dry and rodded separated aggregates is 1: 4: 2.7.

The next step is to determine the volume occupied by 4 cu. ft. of sand and 2.7 cu. ft. of coarse aggregate after they are mixed. This can be done most conveniently as follows:

Determine the weight per cubic foot dry and rodded of each of the separate aggregates. This information can be obtained from the bulking test just described, if the volume occupied by the dry and rodded materials is determined. To obtain the unit weight of the mixed aggregate (dry and rodded) mix a sample in the proportion of 4 volumes of sand and 2.7 volumes of coarse aggregate, rod these into the measure in three layers as before, and weigh. Assume the following data were obtained from these determinations:

Unit weight of sand, dry and rodded	= 112 lb. per cu. ft.
Unit weight of coarse aggregate, dry and rodded	= 110 lb. per cu. ft.
Unit weight of mixed aggregate, dry and rodded	= 123 lb. per cu. ft.

The total weight of 4 cu. ft. of sand and 2.7 cu. ft. of coarse aggregate is 745 lb. $(4.0 \times 112 + 2.7 \times 110)$. This equals 6.1 cu. ft. of mixed aggregates $\left(\frac{745}{123}\right)$. The mix expressed in terms of dry and rodded volumes of mixed aggregate is, therefore, 1: 6.1.

The next step is to find the fineness modulus of the separated aggregates, from which the fineness modulus of the mixture can be calculated.

The results of sieve analyses assumed for this problem are as follows:

Sieve No.	Amounts Coarser than Each Sieve Percent by Weight: Sand	Coarse Aggregate
100	98	100
48	90	100
28	58	100
14	36	100
8	20	98
4	3	97
⅜ in.	0	19
Fineness Modulus*	3.05	6.14

The fineness modulus of the mixed aggregate is calculated by multiplying the fineness modulus of the fine aggregate by the percentage of fine in the combined aggregate and adding to this value the product of the fineness modulus of the coarse aggregate and the percentage of coarse. The percentage of sand in the mix is $\frac{4}{4 + 2.7} = \frac{4}{6.7} = 60$ per cent. The percentage of coarse aggregate is, then, 40 per cent.

*Sum of the percentages in the sieve analysis, divided by 100.

Therefore, the fineness modulus of the mixture is:

$$0.60 \times 3.05 + 0.40 \times 6.14 = 4.28.$$

The proportions may now be stated as 1 volume of cement and 6.1 volumes of mixed aggregate measured dry and rodded, having a fineness modulus of 4.28. These are basic values and can be reproduced with different materials.

Suppose now, for example, that new aggregates having the following characteristics are received:

Sand

Bulking due to moisture and method of measurement, per cent	30
Unit weight, dry and rodded, lb. per cu. ft.	105
Fineness modulus	2.40

Coarse Aggregate

Bulking due to moisture and method of measurement, per cent	5
Unit weight, dry and rodded, lb. per cu. ft.	108
Fineness modulus	5.90

The proportions in which to mix these aggregates to produce a fineness modulus of 4.28 are calculated from the following simple relation:

$$p = 100 \frac{A-B}{A-C}$$

where $p =$ volume of the fine aggregate expressed as per cent of the volume of fine and coarse measured separately;
$A =$ fineness modulus of coarse aggregate;
$C =$ fineness modulus of fine aggregate;
$B =$ fineness modulus of the mixed aggregate.

Substituting, $p = 100 \frac{5.90 - 4.28}{5.90 - 2.40} = \frac{1.62}{3.50} = 46$ per cent; the percentage of the coarse aggregate is $100\text{-}46 = 54$ per cent.

The volume of separated aggregates required to produce 6.1 volumes of mixed aggregates is determined as follows:

Determine the weight per cubic foot, dry and rodded of these aggregates mixed in the proportions of 46 per cent fine and 54 per cent coarse. Assume that this is found to be 125 lb. per cu. ft. Therefore, 6.1 cu. ft. of the mixed material weighs $6.1 \times 125 = 760$ lb. The weight of 0.46 cu. ft. of sand and 0.54 cu. ft. of coarse aggregates is 106 lb. $(0.46 \times 105 + 0.54 \times 108)$. Then 7.2 cu. ft. of the separated aggregates measured dry and rodded in the proportions of 46 per cent fine and 54 per cent coarse will be required to produce 760 lb. $\left(\frac{760}{106}\right)$. Therefore, the volume of sand is 3.3 cu. ft. and the volume of coarse aggregate is 3.9 cu. ft., giving a mix in terms of separated materials dry and rodded of 1: 3.3: 3.9.

To express these proportions in terms that will be used in the plant, the calculated volumes of aggregate must be corrected for bulking.

In this case, the sand was found to bulk 30 per cent over the dry and rodded volume and the coarse aggregate 5 per cent. The volume of sand measured damp and loose which will be the equivalent of 3.3 cu. ft. measured dry and rodded is, therefore, 130 per cent of 3.3 = 4.3 cu. ft. Similarly, the damp and loose volume of pebbles is 4.1 cu. ft. and the mix which should be used with this new material, expressed in terms of plant conditions is 1: 4.3: 4.1.

It should be pointed out that the value of fineness modulus of aggregate which produces the best results depends chiefly upon the maximum size of the aggregate, the larger sizes allowing the use of the higher values. In general, it is most economical to use as high a value of fineness modulus as plant conditions permit.

The following table gives approximately the best values of fineness modulus for aggregates of different sizes:

Range in Size of Aggregate	Fineness Modulus
0—No. 8	2.75
0—No. 4	3.45
0—3/8 in.	4.20
0—1/2 in.	4.60
0—3/4 in.	5.00

These will be found helpful in establishing a starting point for the experiments to determine plant "constants."

Design of Reinforced-Concrete Circular Bins for the Storage of Cement.

By H. A. Ward.*

Introduction.—Circular concrete bins for the storage of material have been used in Europe for the past thirty years, chiefly for the storage of grain. Engineers in this country were loath to accept this type of construction, but the ravages of fire upon the wooden grain elevators and storage houses forced them to adopt a more fire-resisting building material. Steel, tile reinforced with steel rods and brick reinforced with steel rods have all been used in the construction of storage bins, but the advent of sliding forms gave reinforced concrete a chance to forge ahead of the other types of construction. Its lower cost and better fire-resistant qualities have by now practically eliminated other forms of storage bins. These silos that at first were pressed into use for housing grain have now been widely adopted for the storage of portland cement. The first bins of this nature were constructed for the Illinois Steel Co. in 1901 which were followed about eight years later by an installation for the Atlas Co. at Northampton.

Scope.—It is the purpose of this paper to set forth a method of design that may be used for the structural members of circular bins where the depth of bin is at least one and one-half times the diameter. Only those features particularly pertinent to this subject will be brought out here. The requirements of design as set forth in the report of the Joint Committee will be held to govern for the remainder of the design.

Method of Design.

(a) *General Assumption.*—Portland cement comes within the class known as semi-fluids. It exerts lateral pressure and downward vertical pressure but not upward pressure. The cement contained in a deep storage bin is not an unlimited mass such as is assumed when designing retaining walls for earth pressure but is restrained by the walls of the silo.

In seeking a method of design it is only natural that we refer to the action of some other member of this family of semi-fluids, such as grain, where longer acquaintance has given greater knowledge, keeping on the alert for characteristics that vary.

H. A. Janssen, of Bremen, Germany, has developed a formula for the design of circular bins which has been extensively used for grain and is equally applicable to cement for determining the lateral and vertical

*Engineer Turner Construction Co., Buffalo.

pressure at any point. This formula is clearly set forth in Professor Ketchum's book on "Walls, Bins and Grain Elevators."

$$L = \frac{W\,R}{\mu'}\left(1 - e^{\frac{-K\,\mu'\,h}{R}}\right) \qquad \textit{(Formula No. 1)}$$

Where:

V = Vertical pressure of cement in lbs. per sq. ft.

L = Lateral pressure of cement in lbs. per sq. ft.

W = Weight of one cu. ft. of cement.

R = Hydraulic radius $= \frac{\text{Area of Bin}}{\text{Perimeter}}$

$\mu' = \text{Tan}\ \phi'$ = coefficient of friction of cement on the bin walls.

K = Ratio of horizontal pressure to vertical pressure at any point $= \frac{L}{V}$.

e = The base of Naperian logarithm = 2.718.

h = Depth of cement at any point.

We will take the weight of packed cement in a bin, W as 98 lb. per cu. ft. and in figuring the capacity of bins we will assume that a barrel of cement occupies 3.8 cu. ft. Conditions as to capacity required, size of building lot or bearing capacity of the soil determine the diameter of the silo and therefore R and maximum h. This leaves μ' and K for which values must be obtained. It is assumed that L and V are constant for all points on a horizontal plane. This is not quite true, as they will be constant for all points on the surface of a dome.

(b) *Action of Cement in Bins and Pressures Expected.*—The values of μ' and K can be obtained only from experiments. While a great many tests have been made on grain to establish these coefficients, experiments on cement have been very few. From those tests that have been available it appears that cement when first discharged into a bin from the mill flows out so that the surface forms a curve having a radius of about 78 ft. 0 in. or an angle of repose of 6 deg. at the bin wall. This angle of repose changes as the cement stands in the bin and in this respect it is unlike other semi-fluids. Gilbert & Barth's experiments in 1906 on a small bin 3 ft. square and 4 ft. high showed considerable falling off in the lateral pressure after the cement had stood for an hour or so. Other tests have shown an angle of repose as high as 40 deg. If we let ϕ = the angle of repose, an approximate value of K may be expressed by $K = \frac{1 - \text{Sin}\ \phi}{1 + \text{Sin}\ \phi}$. This value was determined by Gilbert & Barth to be 0.4. The depth of bin, condition of bin walls and varying angle of repose will give fluctuating values for K, but in general K is a maximum when ϕ is a minimum, and a minimum when ϕ is a maximum. By referring to experiments on fine grains, sand and ashes, we note that ϕ', the angle of friction of these materials on concrete walls, follows very closely but a little behind the angle of internal friction ϕ and therefore we may assume that for cement ϕ' will be somewhat smaller than ϕ.

The problem that now confronts the designer consists in selecting from this variation of from 6 deg. to 40 deg. a value for ϕ that will produce a safe design for all members under all conditions.

On Fig. 1 may be seen the curves representing three different designs for a bin 32 ft. 0 in. in diameter and 80 ft. high. Case No. 1 assumes an angle of repose ϕ of 6 deg. with ϕ', the angle of friction on the bin wall, as 5 deg. This gives $K = 0.81$ and $\mu' = \tan$ of $\phi' = 0.09$. The depth of cement in the bin is shown in feet on the heavy vertical line of the left

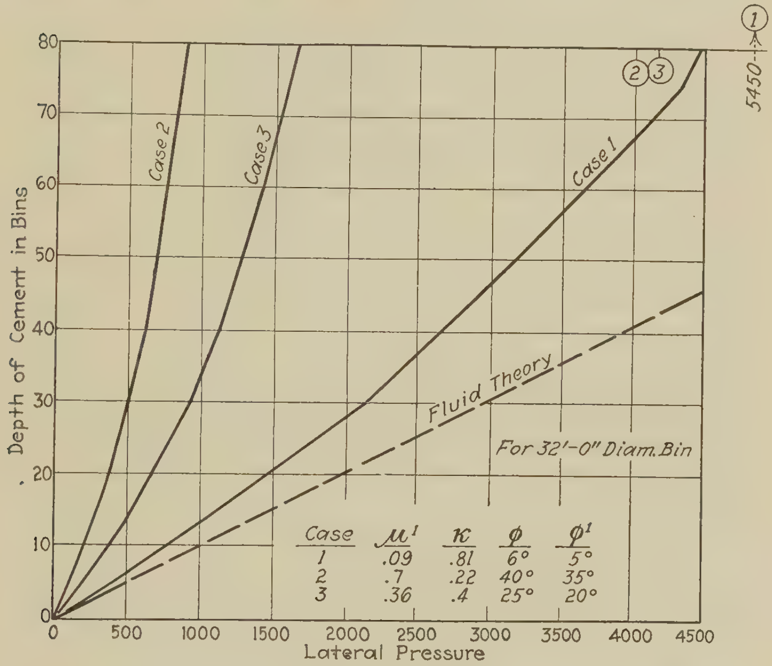

FIG. 1.—THREE DESIGNS FOR BIN 32 FT. DIAMETER, 80 FT. HIGH.

while the horizontal line represents the lateral pressure. Substituting the above values in Janssen's formula produces lateral pressures for various depths which when plotted give the curve shown here for Case 1. The vertical pressure at any point $= \frac{L}{K}$. This vertical pressure on the bin bottom when the bin is full has been indicated on the 80 ft. horizontal line by the number of the case within a circle. Thus for Case 1 the horizontal pressure at the bottom of the bin is shown as 4420 lb. per sq. ft. and the vertical pressure as 5450 lb. per sq. ft. which is lower than the total weight of a column of cement 1 ft. 0 in. square and 80 ft. high by 2390 lb. This difference between the actual dead-weight of the cement and the

vertical pressure on the bottom is accounted for by the fact that some of the dead-weight of the cement is transferred into the bin wall itself before reaching the bottom of the bin. The amount of load carried by the bin wall is equal to the total lateral pressure against the wall multiplied by the coefficient of friction of cement on the bin wall. If $P =$ the total lateral pressure on a section of the bin wall one foot long for its entire height then P μ' will be the load carried by this one foot section of wall and from Janssen's formula we have:

$$P = \frac{W R}{\mu'} \left[h - \frac{R}{K \mu'} + \frac{R}{K \mu'} \; e - \frac{K \mu' h}{R} \right] \qquad \textit{mula No. 2)}$$

P μ' multiplied by the perimeter of the bin will be the total load carried by the bin wall and if this be subtracted from the total weight of the cement and the result divided by the area of the bin, we have a pressure which is the same as the one previously worked out by dividing L by K.

In the same way we have plotted on Fig. 1, Case 2 a curve for this same bin with values for μ' and K corresponding to a condition where the cement has an angle of repose of 40 deg. The lateral pressure at the bottom for Case 2 is 1120 lb. and the vertical pressure is 4000 lb.

The lateral pressure in Case 2 is very much less than in Case 1 and we might expect, therefore, that the vertical pressure on the bottom would be greater in Case 2 than in Case 1, but this is not so. The cause in the decrease in lateral pressure lies in the fact that the angle of repose has changed, that there is more cohesion in the mass and it therefore has greater ability to stand up in steeper piles. This very increase in the angle of repose carries with in an increase in the angle of friction, and while the lateral pressure is falling off μ' the coefficient of friction is increasing so that the net result is a greater load carried by the bin walls and a smaller vertical pressure on the bottom.

In designing a storage bin with the greatest economy and at the same time providing proper strength for maximum stresses it is obvious that neither Case 2 nor 1 is applicable but somewhere between these curves we can find a correct solution.

The dash line on Fig. 1 represents lateral pressure figured on the fluid theory. The curve of Case 1 approaches this line and represents the pressure of the cement immediately after filling. Early designs on steel bins made by the fluid theory failed by crippling of the side walls as sufficient material was not allowed to take the compression caused by the load carried by the wall. In a deep bin requiring four or five days to fill, the angle of repose of the first day's run has increased to almost its final degree by the time the filling reaches the top of the bin. The maximum lateral pressure, therefore, cannot be as in Case 1 or even approach it but would lie closer to the curve of Case 2. During the period of storage, the pressure would be represented by Curve 2, but when the drawing off of the cement is begun the angle of repose is again changed and the lateral pressure increased. Grain experiments have shown that discharg-

ing of grain has raised the lateral pressure but not materially affected the vertical pressure and that the location of the discharge opening varied this increase in lateral pressure. Some lateral pressures where the spout opening was at the wall were increased four times their static pressure. We may expect results somewhat similar to this, although not so great in cement where openings are located at several different points in the bin bottom.

With these points in mind, we have selected the following values for ϕ, ϕ', μ' and K for producing a satisfactory design, both as to economy

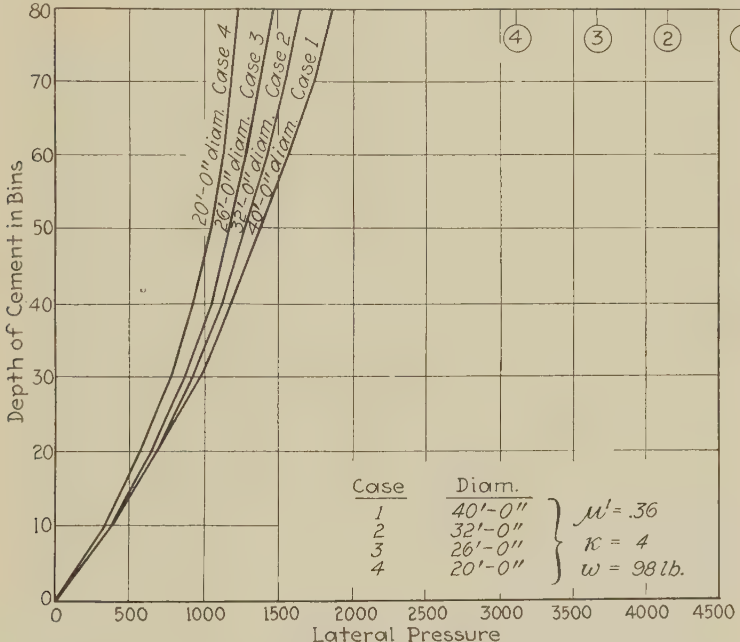

FIG. 2.—LATERAL PRESSURE CURVES FOR VARYING DIAMETERS.

of material and safety under a changing condition within the material itself:

ϕ = 25 deg.
ϕ' = 20 deg.
μ' = 0.36
K = 0.4

If these values are used in Formula 1 the lateral pressures will plot a curve as shown for Case 3 on Fig. 1 with a vertical pressure of 4130 lb. which is less than Case 1 but greater than Case 2. By following this curve it may be noted that the increment in the lateral pressure is great-

est in the first 20 ft. of depth, thereby providing for maximum loading conditions in the upper part of the bin without penalizing the lower portion of bin wall for a condition that does not exist when the total lateral pressure is a maximum.

Fig. 2 shows lateral pressure curves for bins 20 ft., 26 ft., 32 ft. and 40 ft. in diameter with vertical pressures on the bottom shown on the 80 ft. horizontal line based on values for μ' and K as 0.36 and 0.4. This shows that the smaller the diameter the greater is the proportion of total contents carried by the bin walls, so that if the diameter were increased indefinitely we would have an unlimited mass where the lateral pressure curve would become a straight line and the problem would be reduced to a question of retaining walls and K would then become $\frac{1 - \sin \phi}{1 + \sin \phi}$. With the above values for μ' and K we may now proceed to a consideration of the design of the structure itself.

(c) *Design of Circular Bin Walls.*—From Formula 1 we may obtain the lateral pressure at any depth in the bin. If W is in pounds and R and h in feet, L will be the unit lateral pressure in pounds per sq. ft. Consider a horizontal section of the bin 1 ft. deep just above the bin floor. Divide this section on a diameter and consider only one-half of the section which is in the form of a semi-circle. The total thrust on this semi-circle tending to cause lateral motion in one direction is L, the pressure in pounds per sq. ft. times D, the diameter of the bin in feet.

If we apply a force equal and opposite to $L\ D$ the structure will be in equilibrium. There are two points of application of this force, one at either end of the semi-circle where the bin wall was cut by the diameter. The half of the section which we have not considered supplies this resisting force transmitted through the horizontal reinforcement. Then the stress in this reinforcement for a section 1 ft. high for one intersected wall only $= \frac{L\ D}{2}$ and this divided by the allowable unit stress in reinforcement gives the area of steel required in a strip of wall 1 ft. high. When this method is repeated at five-foot intervals to the top of the bin, we have the design of a series of bands of reinforcement for horizontally supporting the walls. Owing to great changes in temperature caused by the deposit of cement at very high temperatures and to the increase of lateral pressure by discharging from spouts near the side walls, the allowable unit stress on reinforcement should not exceed 18,000 lb. per sq. in. Where splices are made the steel should be lapped 60 diameters of the rod, or if flats are used 60 diameters of a rod with equivalent area. Vertical reinforcement consisting of ⅝ rods should be placed about 2½ ft. apart to act as distributing steel and tierods. Where jackrods are used for sliding forms one vertical rod may be omitted for each jackrod. The steel is placed in the center of the wall.

Consideration must be given to the thickness of the wall as much of the load is carried on this shell. In this connection a unit stress of

325 lb. per sq. in. should not be exceeded for the compression on the concrete. This, together with a possible increase in load while the cement is in storage, leads to the following formula for minimum thickness of wall which is derived from Formula 2 by substituting values for μ' and K based on an angle of repose at 40 deg.:

$t =$ not less than 7 in.
$t =$ not less than

$$\frac{R\left[h - \frac{R}{.154} + \frac{R}{.154}\, e - \frac{.154\, h}{R}\right]}{40} \qquad \textit{(Formula No. 3)}$$

Where R and h are expressed in feet.

Openings through bin walls that sever rings of steel reinforcement must have special attention and additional steel is to be provided below and above the openings to take the place of the steel that has been cut. This becomes especially important in the type of storage bin where the floor rests on ground and a conveyor tunnel pierces the bands of reinforcement. Here it is necessary to pick up the severed rings of steel and anchor them securely to a vertical girder transmitting the tension in these rods to a point above and below the tunnel opening. At these points additional horizontal ring steel tie the ends of the vertical girder on one side of the tunnel to the ends of the corresponding girder on the opposite side of the tunnel.

(d) *Design of Bin Floor Slab.*—Two types of storage bins are in use today, one wherein the main portion of the floor rests directly on the earth and a conveyor tunnel is used to withdraw the cement. The other form and the one most in use consists in carrying the bin floor at a sufficient height above a working or conveyor floor to spout the cement from the bins into a series of parallel conveyors. In the first type after we have satisfied ourselves that the earth has sufficient bearing capacity to carry the load we are concerned with only that portion of the floor coming over the tunnel which after we have determined the external load becomes a matter of standard design.

In the second case we have the bin floor supported on longitudinal walls spaced usually from five to eight feet apart. The design of the slab is made as for ordinary construction using a bending moment of 1/12 wl for continuous spans. The bin outlet castings require large openings in the slab reinforcement and care must be taken to distribute the load with reinforcement onto the slab reinforcing steel. This same outlet cuts away concrete often needed in compression and therefore compression steel is required.

In either type the load on the bin floor in pounds per sq. ft. is $V - \frac{L}{K}$ or the total weight of cement minus $P\,\mu'$ times the perimeter of the bin divided by the area of the bin.

(e) *Design of Foundations.*—From Formulas 1 and 2 we may obtain the portion of load carried by the bin wall and by the floor slab. This, together with the dead-load, is transmitted either directly into the ground as in the case of the type of bin whose floor rests on the soil, or through foundation walls onto spread footings in the case of bins with a basement. The foundation walls are of two kinds: First, the circular walls which are a prolongation of the bin walls. These take the weight of the walls and a portion of the load from the bin contents together with a small section of the bin floor slab. They transmit the load down to the spread footing as a bearing wall and do not require reinforcing except such as would be ordinarily required for temperature stresses.

The second kind of foundation walls consists of the longitudinal walls supporting the floor slab. Where the maximum unit load on the ring wall footings does not exceed the bearing capacity of the supporting soil the treatment of these longitudinal walls is in no way different from the circular walls. Usually either the enormous load due to the height of the bin or the poor bearing capacity of the soil requires that these longitudinal walls act as a distributing system to transmit the load over the whole area occupied by the bins. When this is the case these walls are to be figured as girders spanning from one circular foundation wall to the next. As it is necessary to have access from one bin to another, openings must be left in these walls preferably in the center of the span where least damage will be inflicted to that concrete taking shear. The opening must be kept down a sufficient distance from the top of the girder to allow the tension rods to act and far enough up from the bottom of the girder to give sufficient compression without requiring too much compression steel. Attention must be paid to these walls, as they act as a balance wheel to the structure for change in loading due to the variation in the angle of repose of the cement.

The spread footings under the ring walls and the longitudinal walls are handled as in standard construction.

In the first filling of a group of these silos it is important to fill all silos gradually and evenly to prevent undue settlement and also cracking of walls from the heat of the fresh cement.

Conclusions.—In closing this paper the following points are left for the consideration of those who in the future will conduct experiments in connection with this problem:

1. An experiment should be conducted on an actual silo beginning with the filling of the bin and continued for a period of two months after the silo has been filled or until the cement has cooled throughout.

2. Readings should be again taken before the cement is drawn off and continued throughout the emptying of the bins.

3. This experiment should establish values for ϕ, ϕ' and K and as these values will vary as the bin is filled care should be taken that the value of each function is derived from readings taken at the same time.

4. The records should show the time that readings are taken and the depth of the filling in the bin. Also the time when the cement starts and stops flowing into the bin. The diameter of the bin, the temperature of the cement and the degree of smoothness of the inside of the bin wall should all be noted.

5. Experiments should be made on bins that have been filled and emptied at least twice previous to the time of conducting the test in order to avoid possible interference from caking of cement sometimes experienced in new concrete bins.

6. In making a series of tests, bins should be selected that have different hydraulic radii.

DISCUSSION.

Mr. Dockstader. E. A. DOCKSTADER.—You said you limited the concrete stress, I believe, to 325 lb. in compression in bin walls, to take care of increased loading as the bin was filled. I would like to know what the computed stress under maximum load was; also when the cement is hot, what is its temperature. Is there not considerable stress in your circumferential or horizontal steel, due to the difference in temperature between the inside and outside of the bin wall? If the steel were placed near the outside of the wall, would it not act to better advantage to resist the stress due to temperature? What effect, if any, would such a position of the steel have on its ring action in resisting the bursting pressure?

Mr. Ward. H. A. WARD.—In answer to the first question, the limit was placed on that not so much as the increased loading of the outside walls, as to keep that stress down. We are pouring a thin shell with sliding forms. Occasionally a pocket occurs in that bin form, or the concrete is very irregular. Now other things happen in connection with the pouring of those walls, and we feel that the tendency is to go to a larger diameter bin now rather than a small diameter, so we are really erecting a very thin shell at enormous height and we prefer to keep the stresses in that wall down rather than high, through an excessive thickness in the shell, by keeping this down to 325 lb.

The second question with reference to the temperature of the cement—you have nothing as high as you would find in a chimney structure. Probably 200 deg. is the temperature. The temperature varies; it would be anywhere from 120 deg. to 300 deg. going into the bin. This high temperature stays in the bin for months, but more in the center of the bin and the heat does not appear on the outside of that bin. It is cooled from the outside in, and undoubtedly there is some tension put on those rods. Now the reason that we keep the steel as near the center as possible and not toward the outside is that in springing the steel around, especially in small diameter bins, there is a tendency for that steel to spring toward the outside, and we are afraid of getting this too near the surface where corrosion will start and immediately you will have popping of the concrete, spalling off of the concrete, and then it is only a question of time how long those bands are going to be useful. Of the two, I would rather see it nearer the inside and stress the steel beyond the 18,000 lb. than I would to get it near the outside and have it rust. That is one of the reasons why I prefer not to go above 18,000 lb. In some bins for grain, they have gone as high as 30,000 lb. on steel.

Mr. Dockstader. MR. DOCKSTADER.—If you move the steel toward the inside of the bin, the stress in the steel from temperature would become less, but the concrete on the outside of the wall would be more likely to crack. I should think, with the temperature you mention, there might be considerable stress in the horizontal reinforcement due to temperature in addition to

the stress due to bursting action, because, as you say, the outside of the wall stays cool while the inside becomes hot; therefore, the inside elements of the wall expand, and if there is no reinforcement near the cooler inside, obviously it must crack whether you see it or not. If you move the steel toward the inside of the wall, you take the stress out of the steel, but it is not effective in preventing cracks on the exterior of the wall.

MR. WARD.—That is a point I was interested in seeing brought out, in case further tests were made. We have had very few tests and what tests have been made have not been made on deep silos. They approach conditions of unlimited mass as where you have a retaining wall and you do not have the restraint of the circular wall. Mr. Ward.

J. W. LOWELL.—In your statement, as I understood it, the smaller the diameter the greater the proportion carried by the walls; did you not mean the smaller the proportion of the diameter to the height, the greater the proportion carried on the walls? Mr. Lowell.

MR WARD.—Of course the higher you go with the bin, the greater the proportion of load that is carried on the wall, as shown in the case of grain bins more than in the case of cement bins. Where the load is half as great you have a chance of running a bin much higher, to 100 ft., and there comes a point in that bin where the pressure on the bottom does not increase at all. After you get up two and a half times the diameter of the bin, your bottom pressure does not increase at all. That means that the load from there on is carried entirely by the wall itself, but the point I refer to is a different condition entirely. Mr. Ward.

MR. LOWELL.—Is not that because of the fact that the ratio of diameter to height has become less, rather than the fact that the diameter is less? Mr. Lowell.

MR. WARD.—That is true, but that does not enter into the problem quite as much. You have to design your bands of reinforcement for every conceivable height from the bottom way to the top so that that position is changing as you go up. Mr. Ward.

MR. LOWELL.—I am afraid I did not make myself clear. Mr. Lowell.

A. C. IRWIN.—As I get Mr. Lowell's thought, it is the same as I have in mind. He wants to bring out the fact that apparently there are two limitations in the design of the bin bottom. One is reached at a certain ratio of diameter to height of the bin, or depth at which there is no change in the pressure on the bottom. That is one of the limitations. The other limitation is that there is a certain lower ratio of diameter to height at which you would have the total weight of the cement coming upon the bin bottom without any relief because of the fraction of the cement on the side walls. Those are the two limitations; you have stated one of them as two and a half, I believe, the upper limit. Do you have a lower limit? Mr. Irwin.

Mr. Ward. MR. WARD.—I stated in the paper one and a half diameters. This paper was limited by a bin whose height was one and a half diameters; in other words, that is the point where the slope of rupture cuts the surface.

Mr. Lowell. MR. LOWELL.—If Mr. Ward wanted to take an empirical amount of pressure on the cement bin, say in relation to a liquid pressure, how much would that be per square foot equivalent to a liquid pressure?

Mr. Ward. MR. WARD.—Lateral pressure?

Mr. Lowell. MR. LOWELL.—Both.

Mr. Ward. MR. WARD.—On an 80-ft. bin, that would possibly run twenty to twenty-five per cent at the top and about thirty-five per cent at the bottom, or rather the other way around.

Mr. Lowell. MR. LOWELL.—In pounds per square foot?

Mr. Ward. MR. WARD.—Well, your increment would be 98 lb. per square foot, for the lateral pressure, if you put it on the liquid basis. Now, instead of that, you design the bottom rings of that wall for an increment of about 20 lb. per square foot lateral pressure, and as you go up, that curve increases so that you are running over 30 lb. per square foot increment; when you approach the top it ranges between thirty and twenty-five per cent of the liquid pressure.

Mr. Irwin. MR. IRWIN.—That is for the side walls?

Mr. Ward. MR. WARD.—Side walls.

Mr. Lowell. MR. LOWELL.—In determining your computations in connection with the angle of repose of cement, you stated that in the filling of the bin the angle of repose was less than it would be later after the cement had solidified. I suppose you took into consideration the difference in the weight of the cement, did you?

Mr. Ward. MR. WARD.—No, I did not.

Mr. Lowell. MR. LOWELL.—Because when it has been filled you do not get 94 lb.

Mr. Ward. MR. WARD.—I did not take that into consideration, for the reason that that condition exists only for a short time and it is not the real problem that we have to face. We have got to take care of it, but once it is taken care of, you can practically forget that first weight of the cement or that first angle of repose. I merely showed it to bring out the extremes, how it ran from an angle of repose of 6 deg. up to 40 deg. after it had been there a few days.

Mr. Jennings. D. F. JENNINGS.—Have you any data on the widest variation in the weight per cubic foot, and have you taken into consideration the difference

in that at different stages of storage, when the cement was first put into the storage as compared with it after it has been in the storage?

MR. WARD.—I haven't anything that would bear directly except that it ran as low as 90 lb. and up to 101 lb. Mr. Ward.

MR. IRWIN.—One further word about the unit stress compression on the concrete and side walls; the limiting thickness of the side walls was not stated, that is the thickness that would allow a good job of compacting concrete in that wall. From the contractor's side, I believe he will say that he can get a good job in a 6-in. wall. Personally I do not feel that there is any good reason for cutting down the unit stress to 325 lb. on concrete that is good for 2,500 to 3,000 or higher in 28 days; and in the second place, if this low unit stress is based on temperature effects, why not design the bin wall for those temperatures, reinforce it and come more nearly to approaching the unit stresses used in the design of buildings, even those of the Joint Committee? Mr. Irwin.

CHAS. E. NICHOLS.—As I understand Mr. Ward, the low unit stress he advocates results from the mental application of a slenderness ratio reduction formula to standard unit stresses of 500 or 650 lb. per square inch, and he has in mind particularly the advisability of using low stresses for large sized bins where the walls are for practical purposes, not walls of a cylinder but essentially straight walls. When one considers a wall of 4 or 5 in. minimum thickness, (I have built similar walls as thin as 4½ in. in the upper sections of 20-ft. diameter bins 80 ft. high), in a bin 35 or 40 ft. in diameter, it does not seem to me quite so much the wall of a pipe; it begins to look more like a partition wall in a building and I would not care to load it up to the maximum stress without applying a slenderness ratio formula. Mr. Nichols.

LOUIS CLOUSING.—I have found that in working with grain bins it frequently happens that distortion to the walls will come due to discharging from one side. We have got some grain bins that I happened to look at a while back that were held in one direction by a conveyor attached to the wall in a certain line; in the other direction there was nothing to hold them. They were oval-shaped bins instead of round shaped at the top, and at the foundation they kept their shape and they cracked about one-third from the top. Mr. Clousing.

Question Box on Reinforced-Concrete Building Construction.

E. D. Boyer in the Chair.

Concrete Mixing Water.

"What steps are being taken on the job to control the water in concrete mix?"

Mr. Howard. J. H. Howard.—I feel that the answer falls into two parts: first, where old-fashioned methods are being used and no definite water gauge is fixed by the charging apparatus, I think that general information and education have brought out the idea that the less water you use the better, but I do not believe that any steps are being taken to control the amount of water in the mix, it is always governed by what the foreman on the job thinks he would like to have in the concrete. In the second place, where a definite, scientific proportioning and mixing of the concrete is attempted, I believe that steps are being taken to control the amount of water that goes into the mix, either by slumps at uniform intervals or by educating and interesting the superintendent, the foreman and the man at the mixer.

Supports for Reinforcing.

Outline methods for supporting flat-slab bars near top of slab.

Mr. Harding. E. C. Harding.—That has been a source of a great deal of trouble on some of our jobs. We used precast concrete blocks carried on bars that theoretically placed the steel in the right position. Often the span is rather long and the steel rather light, consequently the steel is much lower than it should be. This was brought out as a result of an accident that happened on one of our jobs where the steel was exposed, and it was not any too high in the slab. I figured that this is over the compression area. I have noticed contrivances built up of heavy reinforcing bars bent so that the legs will rest upon the depressed position of the slab and continued across, providing support from the depressed slab right where it is needed. Some sort of support on the stub of the column might be worked out in the form of a clamp of some kind. In two or three jobs I have had the steel foreman wire the steel to the column stub, but whether it remains there or not, I do not know.

Mr. Ahlers. John G. Ahlers.—We have found that the only safe way and the only right way to support steel at the maximum height is by the use

of some of the proprietary articles made for that purpose. The cost is a little high, but it is the only way to get a sure and safe job, and we feel that by using these bent sheet metal or wire supports that are fixed and come to the exact height and supply two by four bars around each column head that is not figured in tension, you get an absolute job that you can be sure of and the cost is well justified in its added security.

T. L. CONDRON.—I am very much surprised to hear the first speaker speak so uncertainly of his own work, as to where the top bars are and that there is no real certainty of their being there. He also fell into the error of speaking of these bars being in the compression area. They are not in the compression area if the design is properly made. The most important tension areas in construction in the flat-slab structure are in the top and not the bottom of the slab, and they are not confined to the region over the column head or what he referred to as the depressed portion of the slab; there are deeply important areas to be reinforced for tension in the bottom of the slab and all the way across from one column to another. One of the difficulties we are continually confronted with is that bars are not held in place, but there is no excuse for their not being held in place. Another difficulty arises from the fact that concrete shrinks vertically, horizontally and otherwise. The tendency is to have the concrete wet enough to get into place, which means to have it unnecessarily wet. When these bars are rigidly held in place by concrete blocks or bent iron supports, the bars themselves do not go down as the concrete shrinks in volume. The consequence is that cracks in the green concrete develop over the top of the top bars, reinforcing slabs; and the thicker the slabs, the more certain are these cracks to develop. The only cure that I know of for that unfortunate condition is to have your concrete with as little water as is possible to get your concrete in proper place and to get it of proper character. That is a real cause of the deterioration of concrete structures that are exposed to the weather. If you watch the job you will see the cracks develop as hair cracks and sometimes you will see them developing as large cracks in the thicker structure. Mr. Condron.

C. B. FOSTER.—I might say that in anchoring bars in connection with the column bars, we have a cup made that slips over the top of a reinforcing rod with a hole in each side to take a No. 9 wire, and it makes a permanent connection, one that cannot be tramped down by a workman. Mr. Foster.

J. W. IMMEL.—I am sorry your question is headed flat slab bars, because I think the greatest difficulties in cracks appearing with tension bars is over beams, in the beam and girder construction. I find in looking over most concrete jobs put up by the average contractor, cracks appear right over the beams due to the top bars having no support there and being merely wired up to either light ⅜-in. bars on the stirrup or not wired at all. The electricians get on the job just ahead of the concrete men and after a few days you will find the tension cracks. I think that as much or more consideration should be given to that in beam and girder construction than in flat-slab construction. Mr. Immel.

PATCHING CONCRETE FLOORS.

"What is the best procedure in patching concrete floors in occupied buildings?"

Mr. Grady. J. C. GRADY.—Is this a floor where the finish is placed at the time or at a later date?

Mr. Boyer. MR. BOYER.—The question does not show that.

Mr. Grady. MR. GRADY.—I had not thought of the subject, but I had in mind trying some alumina cement on the repairs of a floor where the finish was placed afterwards. I have repaired some floors but the monolithic floor is beyond me, I do not know how to repair that.

Mr. Boyer. MR. BOYER.—The chair does not think this refers to the discussion of whether you could repair or patch a floor with alumina cement. The question probably is how you patch it with portland cement.

Mr. Ahlers. MR. AHLERS.—I have had occasion in the last year to repair a very considerable area of a factory floor that had become defective. The entire floor was defective in that case, however, from faulty design, faulty placing of materials by the builder, the reinforcing material having been found to be 4½ in. from the surface instead of 1½ in. After a number of years the floor panels throughout that building were failing and the danger was that the floor and even the building, might have to be torn down. However this particular faulty second floor, with about 15,000 sq. ft. of area, has been put into such a condition that it is carrying its load now. One portion was first repaired five years ago and it is carrying its load perfectly. Now the entire area has been repaired by the use of the cement gun. A somewhat complex problem is involved, but we did get entirely satisfactory results with the gun both on the under and upper side of that floor.

Mr. Boyer. MR. BOYER.—Was the surface of the floor worn off?

Mr. Ahlers. MR. AHLERS.—The surface of the floor was worn in places, but the entire top finish, about 2 in. thick, had originally been put on and that was entirely removed without great difficulty.

Mr. Boyer. MR. BOYER.—Was that the same composition as the body of the floor?

Mr. Ahlers. MR. AHLERS.—No, the wearing surface put on was a sand and cement surface, the body of the floor being of gravel concrete. We had to build on to that floor another section of concrete which absolutely had to adhere to the first. I might say in that connection that there was an area of about 3,000 sq. ft. repaired five years ago, by way of experiment. This last year the rest of the floor was timbered up and carried on falsework. To save that building we made these other repairs. In connecting up the old repairs, we found that if we cut off some of the gunite, applied five years ago, it separated below the gunite; in other words, it adhered tighter to the surface of the floor than the concrete was to itself originally.

CONCRETE FLOOR CONSTRUCTION.

"What is the most economical system of floor construction for light loads in hotels and similar buildings, where a flat plastered ceiling is required?"

J. T. BROWN.—Consideration must be first given to the layout, size of panels, etc., common to the location of the building, section of country and the views of the contractor who is to build it. The layout of the building—column spacing, size of panels, number of like panels, etc.—is usually determined from considerations other than structural economy. The structural engineer may suggest a column spacing giving an economical structural layout which would not work well with the layout for room, bars, etc., desired for the typical floor or for the layout of space in the lower stories. The problem on the structural engineer is one of picking the type of construction that he believes best suited for a given layout. Even then be must consider the layout of bathrooms, plumbing fixtures and electrical conduit work. **Mr. Brown.**

I have found that some types of construction are more economical in one city than in another, due to the availability of materials, local labor conditions and the familiarity of local contractors with a particular type of construction. This personal view of the contractor often has considerable weight in determining the construction that is used. Some contractors keep accurate unit costs, but many do not keep them close enough to determine the relative costs of similar types of construction. The contractor may have equipment as well as familiarity with a certain type and be very strong in his assertions of competitive costs of his pet system over another.

Considering these several points, it is hard to say that one particular type of construction is most economical, as each building must be considered a separate problem. In general, however, it has been my experience, that for hotels, some form of ribbed concrête construction is most economical. I do not feel that it can be definitely stated that any one of the several types of ribbed concrete construction is generally most economical. In this, one must consider even more carefully the question of layout, location and personal views of contractors. In a number of instances, I have had designs prepared for several types of ribbed construction for the same building, and found from the figures submitted a considerable variation in the opinion of the several contractors bidding as to the relative economy. On one job I have in mind four designs were considered. One contractor placed them in order of 1, 2, 3, 4; another, 4, 3, 2, 1, and a third mixed them up a bit. In still another case where four types were considered, the design was carried to the extent that complete designs of building were submitted to contractors fór figures. Not more than two contractors agreed as to which was cheaper, and the low bidder who was awarded the contract did not agree with the others figuring. The interesting point about these figures was that of all figures submitted, the differ-

ence between low and high of the four types was less than 1 per cent of the total cost of the building.

I know of another case in which the contractor was picked in advance of the design—he had a financial interest in the building. The question of construction type was carefully gone into by him and the building built of solid slabs and beams and grids, two-way solid slabs where possible and one-way slabs, spanning rooms 9 and 10 ft. wide, with beams on partitions. I have considered this on other buildings and have not been able to find it as economical as he figured it.

Mr. Lord. ARTHUR R. LORD.—The question is too indefinite to answer definitely because in addition to what Mr. Brown mentioned, the height of the building, the number of stories, its importance and the building code requirements must be considered. In the city of Chicago, for instance, the ribbed system is not recognized; whereas in Cleveland it leads all others in economy. I understand it has a coefficient so very much lower than anything that is recognized in other classes of construction as to give it an unquestionable advantage over another type—the two-way joists of tile and filler.

The height of building has a very marked effect because the difference in type is frequently due to a difference in the dead-load, and in a high building that becomes important, whereas with only two or three stories, it may be negligible. Taking the ordinary case of an eight- or ten-story apartment hotel, we have found that the cheapest type of construction in Chicago is the tile and concrete joist for a moderate span up to 6 ft. Beyond that point we have found the open-joist construction to be the cheapest to carry the same load. Another point in that construction must be considered—the effect on the plastering. With the tile and concrete joist construction you will have a discoloration of plaster over the tile. That is objectionable, as it increases the cost of decorating the building. With the open-joist and suspended ceiling, you get away from that local discoloration. Some types of open-joist construction, however, have the same objection as the concrete joist, in that the concrete is plastered directly.

Mr. Brown. MR. BROWN.—I would like to add that one of those floors I mentioned was a two-way tile concrete system in which I had competitive bids that came within 1 per cent of the total cost, and it was not the cheapest in this particular case.

Mr. Nichols. CHARLES E. NICHOLS.—Mr. Ahlers, will you discuss the proper care of cement bags on the job?

Mr. Ahlers. MR. AHLERS.—There is only one way to handle bags, and that is to do it the right way, take good care of them from the moment they come in, if you want to save money. Though I am talking as a contractor, I think that goes for the cement companies too. What I have always tried to do from the time I was a superintendent myself was to make the men take the bags when they put the cement in the mixer and put them in a dry place. Then you will never have any trouble. Keep them wrapped up. There is as much difference between the way bags are handled on the

job as there is difference between superintendents. There is no money in doing anything to the bags after the cement is once put in the mixer; it costs more money and labor either to shake them or get the cement out of them. You save nothing to speak of in freight and you do not get enough cement to pay for the time. The sooner you can get them bundled up and off the job after you get the cement out of them, the cheaper it is for the contractor.

S. C. HOLLISTER.—I understand this question to relate to the proper care of cement bags on the job. I am speaking now of the cement you get from the bag shaker after the sack is presumably empty. The character of that cement is such that on the average job unless there is a considerable amount of mass concrete the cement shaken from the bags after it has been previously emptied is not worth while. The cement shaken by means of a bag shaker weighs about half as much as ordinary cement; it is very coarse and quite lumpy. Care should be taken in the use of this cement on any work that is not strictly mass work where mixers are permitted. **Mr. Hollister.**

ACCIDENT PREVENTION.

How are superintendents and foremen to be interested in accident prevention?

MR. GRADY.—The superintendent must keep his foreman awake at all times to the prevention of accidents. Two of the biggest factors in preventing accidents are frequent inspection by the superintendent or his foreman. On large jobs it is well to employ a special man to go around twice a day and see that all guard rails and safety appliances are in place. He can also get valuable cartoons from the National Safety Council to display on tracks and shanties. **Mr. Grady.**

H. S. WRIGHT.—There is one method of interesting the superintendents and foremen that I always found rather effective. Your insurance rating of course depends on your freedom from accidents, and especially major accidents. You get the same rate, perhaps, in insurance, but there is a discount from most of the mutual companies for contracting organizations that have the best rating. By placing the superintendents and foremen responsible on a bonus basis, it is often possible to get them so worked up about systems of protection and safety devices as to tend to reduce injury. **Mr. Wright.**

MR. HARDING.—We have gone into the safety idea extensively in the last year and a half and are still looking for information. Our method of attack is along the line of interesting the foreman in preventing the accident himself. One method has been by the use of posters which were obtained from the National Safety Council, *Safety News*, or our own homemade posters. Another stunt we have just inaugurated is an accident report that must be filed with our home office on every accident, no matter how minor. One of the questions on this report asks the foreman how the accident could be prevented. This is one means of interesting him—you are asking him for his advice against future accidents. **Mr. Harding.**

Mr. Gould. H. J. GOULD.—One of our superintendents called up one day and said that he was not allowed anything in his budget for barricading openings. I believe that the point was well taken and there should be a definite item in the superintendent's budget for that particular thing. I think it will forcibly bring it to his attention and save having it come out of the general expense items which are sometimes heavy enough anyway. Another suggestion I saw on this accident prevention was in a state where there was no monopoly insurance, a mutual company was formed of several superintendents and the company, the company putting in half the capital and the superintendents the other half as a working nucleus, and then sharing the savings among the superintendents in proportion to their accident ratio.

Mr. Nichols. MR. NICHOLS.—Regular reports that are circulated not only in the home office from various jobs, but that go from one job to another, will help along in that respect. That is particularly true in large organizations, where so many of the superintendents know each other, know the personnel of other jobs, and are really interested in their friends in the work. Our practice for a number of years in accident prevention has been to circulate monthly a summary report of accidents on all active jobs in the organization, its rates, the different jobs and the superintendent's names are given by a definite rating schedule. It gives perhaps a little bit of publicity to the unfortunate man, when he is unfortunate, but I think it materially helps in keeping them all keyed up to the necessity of it.

Mr. Christian. H. A. CHRISTIAN.—I presume that the accidents in question are lost time accidents entirely.

Mr. Boyer. MR. BOYER.—I do not know.

Mr. Christian. MR. CHRISTIAN.—Trivial accidents, such as scratches, etc., are not under consideration. We have those with us all the time, but it is the lost time accident that is the damaging one. I contend that you have to sell the accident prevention from the top all the way down to the men who work on the job. One of our ways for striving to this end is to put up a score board for all the men on the job to see. The different jobs are divided into their component crews and each crew has a horse; in other words, the score board is a horse race. As long as each crew has no accidents, that horse is traveling. Now he can travel for a period of a month, six months, or a year, but just as soon as the crew has an accident, that horse halts in his race. You can imagine the keen rivalry in the whole department, and it is accomplishing results.

HOIST TOWERS.

Discuss wood vs. steel concrete towers.

Mr. Forshee. W. A. FORSHEE.—For the last six or seven years we have used steel towers almost exclusively, and from all the records and data we have been able to gather, we do not believe there is very much comparison to be made between wood and steel towers. The first cost of the steel towers, perhaps,

seems high, but we believe that in about five jobs the towers pay for themselves. We have also found that they are safer in operation than the old type of wood towers. We almost never have any delays due to trouble with the bucket in the tower which we often did have with the old type of wood towers. We find that the erection of the steel towers apparently costs from about half to two-thirds more than to erect wood towers. The salvage, of course, on the wood tower, is practically nil, depending on whether it pays to wreck it or whether we throw it down. We have given up the idea of wood towers almost altogether, except possibly for very low buildings.

MR. AHLERS.—Do you use union iron workers and at what rate per day, for erecting these steel towers? **Mr. Ahlers.**

MR. FORSHEE.—We have erected them both ways at different places, and we have found that where we have a big crew of men familiar with tower erection, the higher-rate men sometimes erect more cheaply than in the smaller towns, where we can use whatever labor we want. **Mr. Forshee.**

MR. HARDING.—We are using steel towers almost exclusively on jobs of any size. We attempted a number of years ago to standardize on wooden towers. This was before the present development of steel towers. We found the tendency among our men was to practically shoot them through full of holes for the connections, and it was a source of endless trouble. The sun would hit them, and dry them out. I am not prepared to state any definite cost on them, but our experience has been that it is cheaper in the long run, taking everything into consideration, to use the steel tower. **Mr. Harding.**

MR. NICHOLS.—I am sorry that our experience does not lead us to the same conclusion. I will agree with Mr. Harding on that, it may be that we became discouraged with our early unsuccessful use of steel towers. It may also be that that was back in the times when the steel towers were sold more on the basis of minimum weight of steel that the contractor could give you than on the basis of safety. I cannot give any definite figures on costs because our experience has been so slight on that, but our experience has led us to lean more to the wooden towers. That, however, is perhaps because most of our large jobs are scattered over the country and it does not pay us to maintain equipment which has to be transported over considerable distance. **Mr. Nichols.**

MR. AHLERS.—Speaking entirely from the standpoint of original cost, depreciation, maintenance, and the labor of erection and dismantling, we have found that the steel towers cost more than the wood towers. Even though we own steel towers, we are gradually getting back to the idea of using wood towers almost exclusively except in cases where it clearly shows that the steel towers are cheaper. That is based entirely on both the total cost of placing the concrete and the plant in and out. **Mr. Ahlers.**

C. B. FOSTER.—We erected a steel tower on a four-story building job. It cost us a thousand dollars to erect the tower. We erected a wood tower for a six-story building and it cost us $500 for lumber and labor to put **Mr. Foster.**

it up and take it down. That was at Indianapolis, and since that date we have developed a tower 50 ft. high that goes on a truck that we can raise in thirty minutes and lower in the same time.

APPRENTICESHIP TRAINING.

How may the interest of superintendents and foremen be aroused in apprenticeship training?"

Mr. Nichols. MR. NICHOLS.—Tell them about it and go around tomorrow and tell them again; put the statistics before them which have been put before a good many contractors' associations in recent years in their discussions and their co-operation with the unions. Show them and keep constantly before them the fact that the supply of skilled mechanics is rapidly falling off as compared with the demand for them. You can only do that by constant repetition. That is something that someone higher up in the organization than the superintendent and foreman has to follow as routine practice. Teach your superintendents or foremen the basic principles that they must constantly keep before them if they are going to maintain the efficiency of their own organization and the industry.

BULK CEMENT.

Use of bulk cement; (a) what savings and economies. (b) how should it be handled on the job?

Mr. Wright. MR. WRIGHT.—This question of bulk cement depends on three major issues: How far have you got to haul your cement? is air available on your job? and how big is your job? The reason for that first question is that sometimes in the case of ocean carriers and particularly on long hauls, the cement solidifies when packed in the car, because the interstitial air spaces are so minute that the air is readily expelled and sometimes the car becomes rather stiff by natural compacting. Sometimes it takes a crew of six or seven men all day to unload 200 bbl. Ordinarily, the haul is so short that that difficulty does not arise. The question of air power comes in on the average large job. Cement arches, bulks and cakes in the bins about the same as it does in the car, and it is often necessary to apply air to it to loosen it. That is done by the insertion of a pipe with 1/32-in. holes through it and a canvass bag over the holes, tied on with wire. About 80 lb. of air are required to force the air up and agitate the cement.

On a big job you can afford to put in a type of handling equipment that is most efficient, whereas on a smaller job you cannot. Now there are a great many real economies in bulking cement. The first and major one, from the viewpoint of the contractor, is that it is so much lower in labor cost. With a short haul and mechanical scoop and elevators, you can unload seven to eight cars a day, with two men in the car and one man around the hoist, whereas on the ordinary job it requires several more men than that. I should say there is a possibility of a saving, counting

both labor and mechanical handling, of 15 to 25 per cent in the unloading of bulk cement. Mr. Wright.

The second saving lies in the fact that most contractors figure about 5 per cent loss on bag cement. About 5 per cent is figured for the cost of shaking, bundling and shipping back the empties to the mill. Now that runs up on a job where there happens to be a sand bag scarcity or some other immediate need of sacks. The contractor will often use a 10¢ cement sack instead of purchasing other sacks in the market. There is also a saving in cement. How real that is depends on how well the man at the mixer drops his cement into it. I have seen a good many of them grab a bag by both corners and hold a pound or a couple of pounds of cement in the bag. That is unusual, but on most jobs the man who is shaking sacks just about pays his wages and the cost of the gear to shake with. There is a real saving of course on the interest on your investment. That saving capitalized at 5 or 6 per cent represents considerable if you ship your sacks once in thirty or forty-five days. If you ship back 2,000 or 5,000 sacks in a shipment and you have carried those—I believe the average contractor today discounts his bills—that means that he has carried the burden of costs on the sacks.

From the angle of the manufacturer there is a real saving too. That saving raises the question of interest on the capital invested in the sacks. The cost of getting those bags back into his shop—repairing them, cleaning them, counting them, the timekeeping and the overhead necessary in that connection—is not gratuitous, and eventually it comes back into the cost of the job. I think that as to the location of the job as to distance of haul, there is not much advantage one way or the other. On large jobs with a long haul it is necessary to build some sort of handling device at the railroad siding or the wharf as a temporary storage. That has got to be charged off against the entire job cost. If a man does that the second portion of this question is, the proper tool for handling it—if a man does use bulk cement. The proper tools are countless. The scoop in the car is perhaps the most efficient means of getting it off the car. Then he probably has to provide a bucket elevator or belt conveyor or some other means of conveying it to his storage bins, and then possibly a weighing device to get it to the job. That requires a frequent test on the cubic capacity of the container used to measure it.

The lining of the bins is one thing that might be mentioned. It is sometimes advisable to line with light sheet metal the interiors of the bins.

Apparently there is not so much damage from leaky cars where bulk cement is used as where sacks are used. That is true because of the property of the cement in settling, also the property it has of gathering moisture and balling up. In bulk cement moisture gets into the car and by capillarity is carried through a whole tier of sacks, whereas in bulk cement, moisture comes in through the top or sides and will trickle in one location, ball up and that ball will continue to absorb moisture for some time. Lumps or scales on top of the load can be easily removed and discarded.

COLUMN HEADS.

Is it possible to adopt certain standard sizes for the heads of round columns? This might make it possible to design better forms for column heads. At present, the diameter of the column head where it intersects the dropped panel, is worked out as shown in Fig. 14, Appendix I of the Joint Committee report, with the result that forks for column heads must be designed to provide for any diameter, where the head intersects the drop panel, to the nearest inch.

Mr. Irwin. MR. IRWIN.—I do not believe there is. That is based on the fact that I do not believe it is possible to use a standard size of column head. If it had been possible, the Joint Committee would have provided such standardization. It all depends upon the economy of the design, and if you standardize such a fundamental thing as the diameter of a column head, you have removed the possibility of reaching the most economical design.

I am not satisfied with the discussion on Question 3, "What is the best procedure in patching concrete floors in occupied buildings?" I do not see that the question of whether or not the building is occupied has a great deal to do with it. You have got to provide a clear space where the patch must be placed. The second consideration is how to go about it. To patch any concrete surface, whether it be a floor surface, a wall or what not, first, clean the surface to be patched of all loose material; second, roughen that surface, if it is not already roughed through the process of cleaning; third, thoroughly soak with water the area to be patched. That means soak it just previous to the application of the patching material. However, there should be no film of water on that surface. Fourth, select for the patching material a cement and aggregate as nearly as possible of the same proportions used in the original slab. Fifth, apply the patching material with force. Throw it against the surface, if you please.

Mr. Boyer. MR. BOYER.—With a gun?

Mr. Irwin. MR. IRWIN.—Oftentimes the area is so small that it is impossible to get a gun action on the job. In that case, simulate as nearly as possible the action of the cement gun, which means that the material should be thrown against the concrete surface. After it is thrown against the concrete surface, do not disturb it. There should be no troweling, screeding or floating after that first contact of material with the old slab. Let it harden. To produce the final surface, use the same method after the first application has hardened. Then you can screed that off with the least possible work to give you the finished surface. Keep the patch wet as long as you can and it will stick and give good service. If you do not believe it, take some surface you do not want to have to clean off, throw some mortar against it and go away and forget it and come back a week or ten days afterwards and try to get it off.

Mr. Hollister. S. C. HOLLISTER.—Returning to Question 10: Because the Joint Committee report gives a minimum size for the column head diameter, it is not

necessarily implied that the column head may not be larger and thereby permit, for the particular job, standardization of the column head. The same holds true in the design of beams or any other members on a given job. It is much more economical of construction to standardize the dimensions of the unit than to design the members for the exact minimum requirements under any code, whether it is a Joint Committee code, a city code, or whatnot; so on any job it is necessary to use common sense in order to introduce elements of standardization for the purpose of economy. And it is not proper to be wholly limited by the minimum requirements that the code may provide, because sometimes it is much more costly than to put in additional material. It is possible to put out standard type of head and still answer the requirements of the Joint Committee report.

Mr. Irwin.
MR. IRWIN.—Does it apply to all buildings?

Mr. Hollister.
MR. HOLLISTER.—I was presuming that you would only build one building at a time with one set of forms, and therefore it would be advantageous to standardize the forms for that particular building. There is no point in standardizing column heads for all buildings no matter what the use or size. The particular point on standardizing is to give the greatest number of uses to a given set of forms and thereby introduce an element of economy. That would refer in most instances to the particular job under construction.

Mr. Irwin.
MR. IRWIN.—In other words, then, he says no, when it comes to the general proposition of standardizing column tops; is that right?

Mr. Boyer.
MR. BOYER.—I think he said yes.

Mr. Irwin.
MR. IRWIN.—In a particular building you can standardize it, but the last thing said was no.

Mr. Boyer.
MR. BOYER.—The question is, is it possible to adopt certain standard sizes for the heads of round columns, and Mr. Hollister says it is possible.

Mr. Irwin.
MR. IRWIN.—For a particular building.

Mr. Boyer.
MR. BOYER.—Yes, he qualifies it.

Mr. Nichols
MR. NICHOLS.—What is Mr. Irwin's interpretation of that certain percentage of sizes? If he means one certain standard size, quite obviously his answer is right. Sizes, however, is plural and admits of interpretation of perhaps a dozen different sizes varying by 2 in., 4 in., or even 6 in. I think he probably qualifies his own statement mentally, if not orally, quite as much as Mr. Hollister. The thought there is that we do not disagree really, we probably agree. Mr. Hollister feels that that can be standardized, and his ideas are perhaps with 2-in. variations. I would agree also even to the extent of saying 4-in. variations in those certain standard sizes. Someone else's interpretation might be something else.

Reports of Committees of The American Concrete Institute

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Twentieth Annual Convention

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Report of Committee E-6 on Destructive Agents and Protective Treatments.

Continuing the work of the past few years, this committee is endeavoring to make a systematic study of various ills to which concrete is subject with a view of isolating the agency responsible for each ill and suggesting if possible, the remedy. It will be appreciated that this is a large undertaking and one that cannot be completed in any single year.

The program for 1924 consisted in the investigation of several groups of structures to which attention had been drawn by different members of the committee and others interested in its work. The structures examined have been exposed for periods varying from 9 to 20 years to the usual severe climatic conditions of the border or nearby states and provinces of the United States and Canada in which they are located. The structures examined were in all conditions of preservation, ranging from those in practically perfect state to those in which extensive repairs are required or are already under way.

The study consisted in a detailed examination of the structures and the collection of available information bearing on the construction. In most cases men thoroughly familiar with the conditions attending the work were interviewed. In some cases, sources of the aggregates were visited and progress photographs examined. The most useful and reliable information was found in the statements of qualified individuals who were connected with the construction work, and by the appearance of the structures. Another source of useful information was the examination of nearby structures or structures of about the same age and under similar exposure which were still in good condition and concerning which data were available. The data were largely collected by the secretary of the committee.

It is the purpose of this report to present typical illustrations of the structures examined and to point out what seems to the committee the significance of the findings and the lessons to be learned.

The committee proposes to continue the study of faulty concrete and urges members of the Institute to contribute to the work by sending in data suitable for use in later reports or suggestions as to examples which offer fruitful field for investigation.

Damage Due to Porous Concrete.—In presenting the results of this investigation, the committee wishes to direct attention first to the impor-

tance of porosity on the durability of concrete. A study of the illustrations will show, as the field examination so clearly brought out, that the damage already done, and the possibility of future damage in these structures is in proportion to the freedom with which water enters or passes through the concrete.

The recognized destructive agents are the freezing of entrapped water with consequent expansion and disruption and the solution of portions of the cementing material by percolating water with the subsequent crystallization, either within the concrete pores adjacent to the surface or on the surface. Because of the location of the structures, the action of frost no doubt played an important part in all of the disintegration found. The solvent action appears to have been limited to those structures through which water readily percolated.

The question is frequently raised as to the relation between disintegration and age. Obviously, time is an essential element in the process of disintegration and where once action has begun it continues as time goes on. There is evidence in some of the structures examined that the rate of destruction increases with time, as each year's action exposes greater areas to attack for succeeding years. However, where the opportunities are lacking, time is of no avail to the destructive agents. The good structures illustrated in this report show no disintegration except at points where there was some porous spot for the action to start.

Evidence of frost action alone is shown in the canal walls of Figs. 7, 8, 9, 10 and 12, which are alternately exposed to water and air, giving ample opportunity for saturation and subsequent freezing. Attention is called to portions of the walls which are unaffected; these show that it was possible to obtain impervious concrete with the materials used.

The effect of percolation is most prominently seen in the illustration of Dam A in Fig. 1 where large areas of the downstream side are heavily coated with efflorescence deposited from the percolating water. In places under these deposits and elsewhere on the surface, the concrete has been disintegrated to a depth of 8 or 10 in. What proportion of this disintegration resulted from the deposition of dissolved matter within the pores and what proportion from the effect of frost, obviously cannot be determined. In this structure also, certain areas are much less affected than others, showing that where less permeable concrete was produced the destruction has been less.

In Dam B, Fig. 2, which is not a great number of miles from Dam A and subjected to the same general conditions of exposure, attention is called to the excellent condition of the concrete with only a slight efflorescence shown at some of the fill planes. The condition of this dam and the presence of this small efflorescence when contrasted with Dam A shows in a most convincing way the relation of porosity to disintegration.

Causes of Porous Concrete.—The committee desired also to direct attention to the causes of porosity in concrete, as an understanding of the

causes points the way to their elimination. Porous concrete in the structures covered by this study resulted from one of the following:

1—Excess of mixing water,
2—Deficiency in cement,
3—Dirt or excess of fine material in the aggregate,
4—Segregation of materials in handling and placing the freshly-mixed concrete.

Of these, excess mixing water was found to be by far the most important as well as the most common. This importance arises not alone because of the direct relation between porosity and excess water, which is serious enough in itself, but because the effect of each of the other agencies is greatly aggravated when it occurs in connection with the use of over-wet mixtures.

The characteristic evidence of over-wet mixtures is the increasing porosity from bottom to top of any section of concrete deposited as a unit, ending at the top in a layer of laitance. The laitance layer and the more porous concrete immediately below offer the first opportunity for the passage or absorption of water. The destructive action begins here and works downward. Dirt or excess of very fine material in the aggregate when used with an excess of water is floated to the top or other surfaces, increasing the porosity and decreasing the resistance to water and frost. In the same way when lean mixtures are flushed into place with an excess of water, they lose a considerable part of the cementing material which collects at the top in the laitance bed.

Contrary to a popular belief, the removal of the laitance layer does not remedy the condition resulting from over-wet mixes. It only removes one part of the weakened structure. The porous concrete immediately below the laitance still remains. Laitance, it should be emphasized, is only a symptom; the real disease is over-wet concrete.

Segregation of the materials in handling and placing concrete is a difficulty that arises almost entirely from the use of over-wet mixtures. Proper consistency combined with correct proportions practically eliminates segregation.

In the illustrations presented here will be seen many interesting manifestations of the causes of porous concrete. The dam in Fig. 1 shows the result of using extremely wet mixtures combined with some deficiencies in cement. This structure is the "horrible example" of what should be avoided in concrete construction. The splendid condition of the portions of Dam B shown in Fig. 2, and of the walls in Fig. 4 and 14 show that such conditions as found in Dam A can be avoided.

Fig. 3 shows another example of porous concrete which has been severely attacked. The bad portion is just below a fill plane, the concrete immediately above and a few feet below being in good condition. This is a portion of a dam which is otherwise in excellent condition, and indicates the importance of constant care in the control of the mixing water if per-

manence in all parts of the structure is to be obtained. Contrast with this the appearance of the main portion of the dam (Fig. 2) which was placed in proper consistency, or the wall in Fig. 4 where excess water was made to flow over the top of the forms by requiring the concrete to be heaped well above the form and allowed to stand some time before finishing.

The breakwater in Figs. 5 and 6 offers a splendid example of the effect of insufficient cement. That portion which was built of 1 : 2 : 4 concrete has stood up remarkably well for 20 years under the extremely severe exposure of a Lake Michigan waterfront, while the portion built two years later with 1 : 3 : 5 concrete has been badly attacked, requiring extensive repairs. Both portions of this wall were carefully placed with a dry tamped mix. It is interesting to speculate on the probable condition of the 1 : 3 : 5 section had the concrete been slushed into place in the wet condition used in some of the other structures illustrated. No doubt both sections would have been improved by the use of a puddling consistency, but with over-wet mixtures, the resistance would have been greatly lowered. The 1 : 3 : 5 section would probably have been a total wreck in 10 years if placed as wet as was the Dam in Fig. 1.

The combination of excess water and dirty aggregate was responsible for the porous concrete which resulted in the severe surface disintegration shown in Fig. 10. The structure in Fig. 13 is in extremely bad condition as a result of using bank-run mixtures with varying sand content and a gross excess of water. Contrast with these two structures those illustrated on the same pages and note the thoroughly satisfactory results obtained by proper care in the quantities of cement and mixing water.

Fig. 16 shows another interesting case of porous concrete due to excess mixing water. Fig. 17 gives a striking comparison between two structures where the conditions were identical except in the care used in construction. Fig. 15 shows two parts of a single structure which plainly show the difference in water content.

Conclusion.—From the study of the data resulting from this investigation, the committee believes that the defects which were observed in the structures studied, were largely due to faulty manipulation of the materials and that permanent structures can be achieved by the production of concrete that is impervious to moisture.

This leads to the second conclusion that the committee has drawn from this study, namely, durable concrete, that is, impervious concrete can be produced by adherence to the following requirements:

> **Clean aggregates of durable minerals; a mixture of a fair degree of richness; the use of a puddling consistency; and care in placing and curing.**

A number of methods for the design of concrete mixtures have been developed within the past few years which differ somewhat in their method of approach to the problem and in the details of manipulation. This

diversity in the methods proposed by recognized authorities has served to confuse many engineers, whose interests and responsibilities in other phases of engineering and construction, as well as limitations of laboratory equipment, have not permitted them to make their own investigations. As a result of this confusion attention has been diverted from the outstanding importance of durability by the emphasis which has been placed on strength and economy in these theories of proportioning. Of course, these factors must not be neglected, but they should not be allowed to obscure the fact that the life of the structure is also an element of ultimate cost, which, in most engineering works, outweighs many times the economies possible by fine distinctions in theories of proportioning. The old rule which requires the use of an amount of cement somewhat in excess of the voids in the aggregate is a good rule when combined with the newer knowledge—that no more water should be used in mixing than necessary to produce a workable puddling consistency.

M. M. UPSON, *Chairman*,
P. J. FREEMAN,
W. K. HATT,
E. VIENS,
G. E. WARREN,
R. B. YOUNG,
F. R. MCMILLAN, *Secretary*.

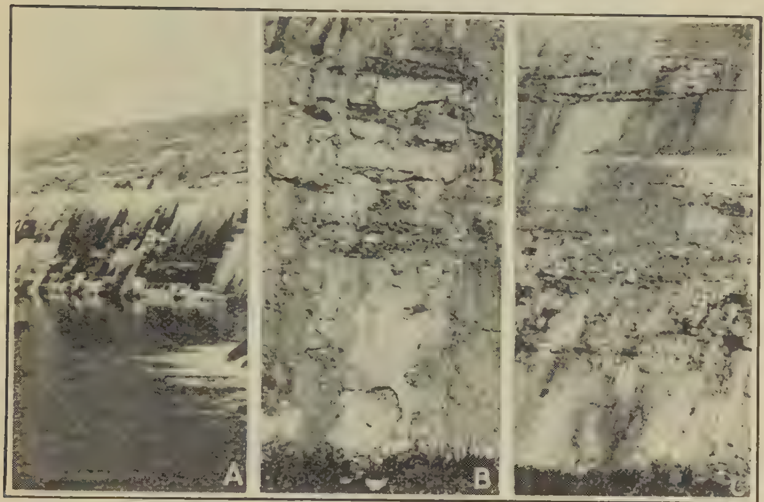


FIG. 1.—DAM A. CYCLOPEAN CONCRETE ABOUT 13 YEARS OLD, ILLUSTRATING THE DESTRUCTIVE EFFECT OF PERCOLATION AND FROST ON POROUS CONCRETE.

Porous concrete due largely to grossly over-wet mixes with some deficiency in cement in portions. Nominal mix of concrete 1 : 2½ : 5. (a) General view of downstream face. (b) and (c) nearer views.

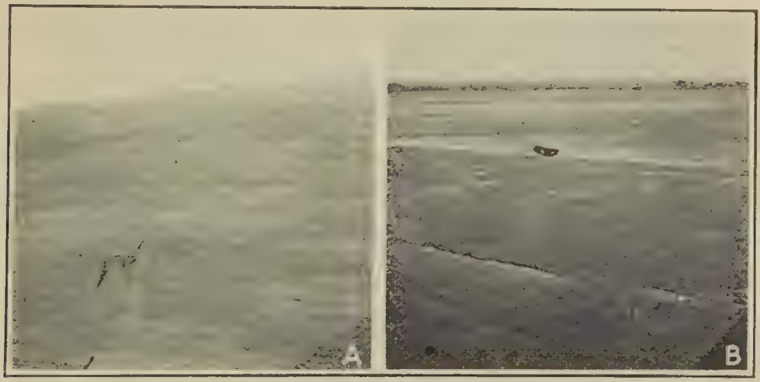


FIG. 2.—DAM B. CYCLOPEAN CONCRETE ABOUT THE SAME AGE AND EXPOSURE AS THAT IN DAM A.

Note the excellent condition of the concrete with only occasional efflorescence at a few fill planes. During the construction of this dam great care was taken to avoid over-wet mixes. Nominal mix of concrete 1 : 2½ : 5. (a) General view of spillway. (b) Close-up of spillway crest.

FIG. 3.—PORTION OF THE HEAD-GATE WALL OF DAM B IN FIG. 2.

Note the similarity of the one spot on this portion with portions of Dam A, Fig. 1. Laitance and porous concrete at the top of a section resulting from the use of over-wet mixtures provides a ready path for the percolation of water. This view shows the only flaw in an otherwise almost perfect structure.

FIG. 4.—CANAL WALL IN PERFECT CONDITION AFTER TEN YEARS OF SEVERE EXPOSURE.

Clean aggregates were used and the fresh concrete was heaped above the top of forms when finishing a section, in order to force out excess water. The wall top is without a crack or flaw.

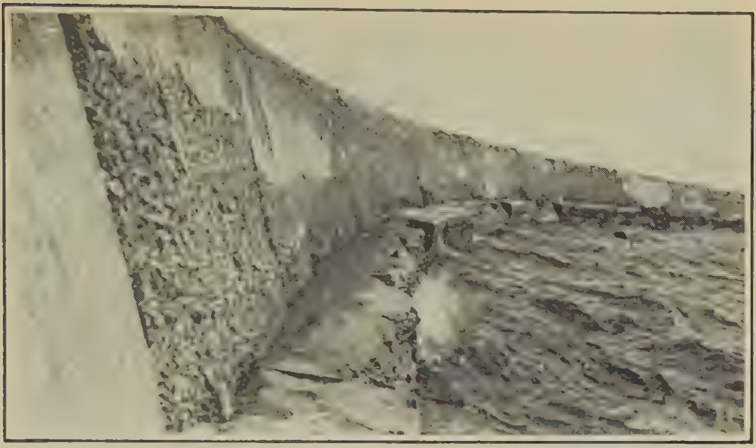


FIG. 5.—BREAKWATER BUILT IN 1906 WITH 1: 3: 5 CONCRETE, ILLUSTRATING THE EFFECT OF WAVES AND FROST ON POROUS CONCRETE, RESULTING FROM TOO LEAN A MIXTURE.

The left foreground shows a portion of the section of this breakwater illustrated in Fig. 6.

FIG. 6.—SECTION OF THE BREAKWATER SHOWN IN FIG. 5.

The section built in 1904 with 1 : 2 : 4 concrete is in excellent condition. The right foreground shows a portion of the 1 : 3 : 5 section illustrated in Fig. 5. Note the greater durability of the 1 : 2 : 4 over the leaner concrete.

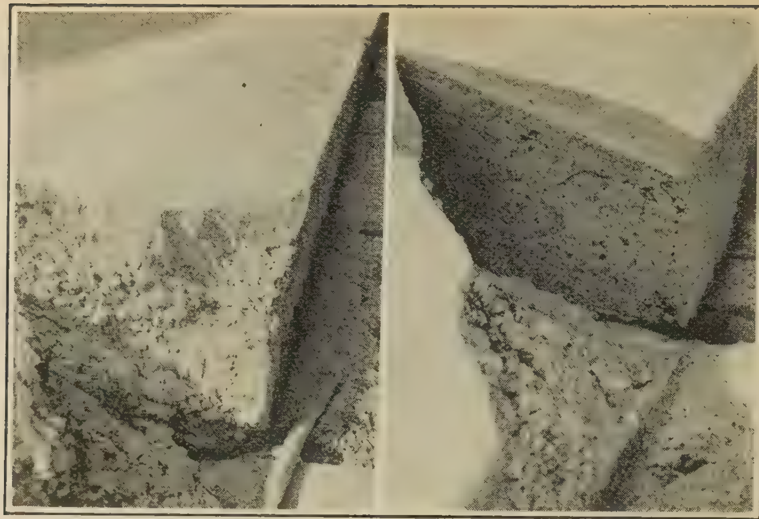


FIG. 7.—CLOSE-UP VIEWS OF THE 1: 2: 4 SECTION OF THE BREAKWATER SHOWN IN FIG. 6.

The destruction begins at joints or any point where water finds access. In the view on the right note the sharp edges, well preserved after 20 years of wave and frost action.

FIG. 8.—A PORTION OF THE 1: 3: 5 SECTION OF THE BREAKWATER IN FIG. 5.

The disintegrated area in the foreground covers an area of about 100 sq. ft. This illustration is typical of the destruction at many construction joints in the 1:3:5 section. In some places where the destruction had progressed much farther the wall has been restored. Contrast this with the views in Fig. 7 where the destruction has progressed only a matter of inches.

FIG. 9.—LOCK WALL IN WHICH OVER-WET MIXES WERE USED.

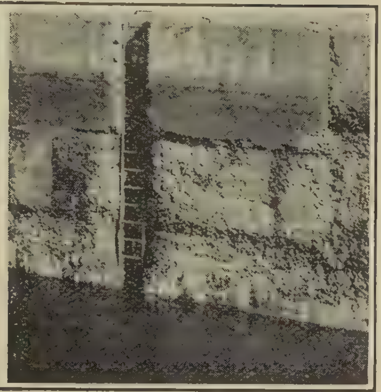
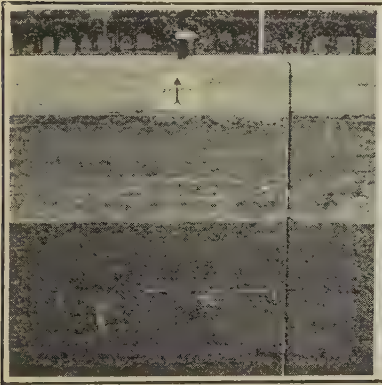
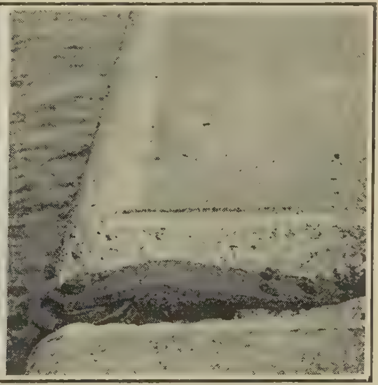


FIG. 10.—LOCK WALL IN WHICH BOTH OVER-WET MIXES AND VERY DIRTY SAND WERE USED.

These structures illustrate the effect of frost on porous concrete. There are portions of both structures which are well preserved and solid. The greatest destruction is found at the fill planes bounding sections placed at different times.





FIGS. 11 AND 12.—TWO VIEWS OF LOCK WALLS IN WHICH CARE WAS EXERCISED IN THE CONTROL OF THE MIXING WATER.

Note the excellent condition of the top in these walls. Exposure and age of these much the same as in the walls of Figs. 9 and 10.

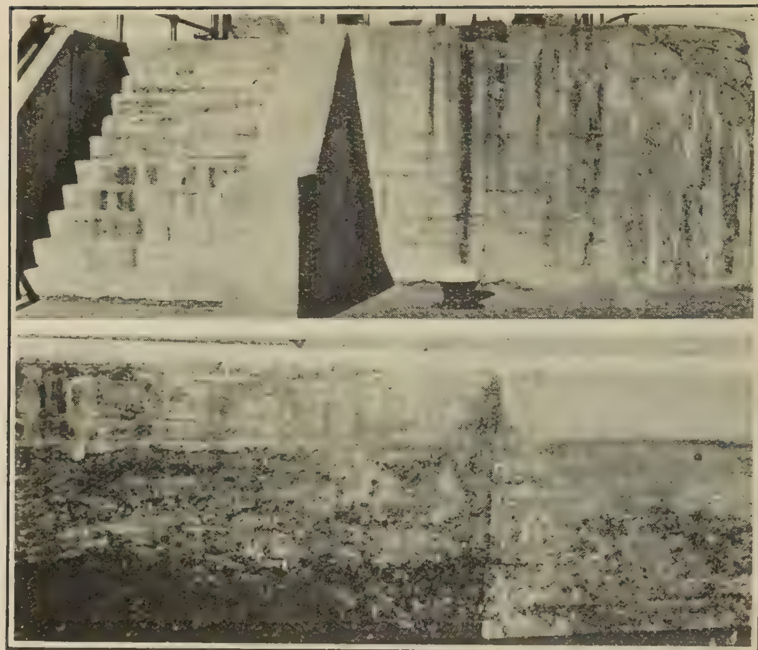


FIG. 13.—TWO VIEWS OF AN EXTREME CASE OF THE DESTRUCTION WROUGHT IN 12 YEARS BY FROST ON EXPOSED POROUS SURFACES.

The concrete in this wall was placed using bank-run aggregate with insufficient cement and excess water.

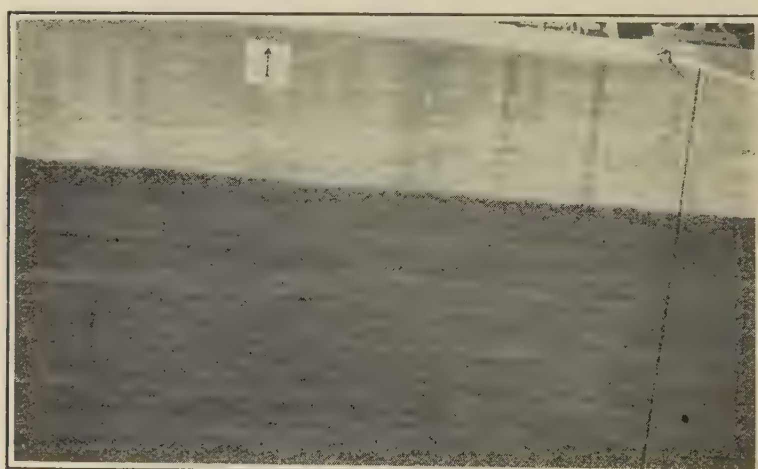


FIG. 14.—AN EXAMPLE OF A WALL IN EXCELLENT CONDITION AFTER 16 YEARS IN ABOUT THE SAME EXPOSURE AS THE WALL IN FIG. 13.

Although built in the winter, the work was so well protected and the concrete so carefully placed that the structure is almost without a flaw at the present time.

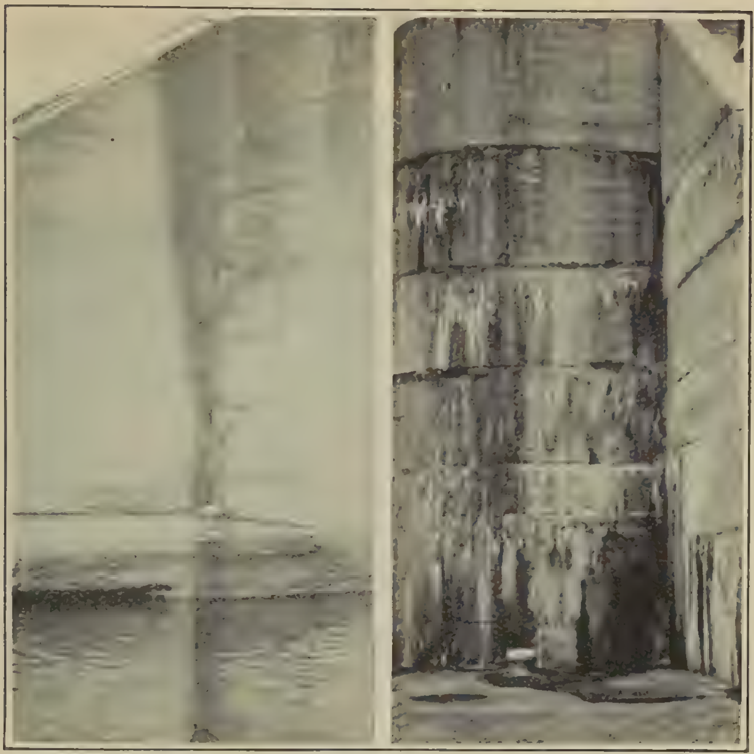


FIG. 15.—TWO WALLS OF A SINGLE STRUCTURE SHOWING PERCOLATION THROUGH LAITANCE SEAMS AT FILL-PLANES AND POROUS CONCRETE IMMEDIATELY BELOW.

Note the better condition in the wall at the left. A little greater restriction in the mixing water and this wall would have been in very good condition.

FIG. 16.—TWO VIEWS OF ANOTHER STRUCTURE SHOWING THE DISINTEGRATION OF POROUS CONCRETE RESULTING FROM EXCESS MIXING WATER.

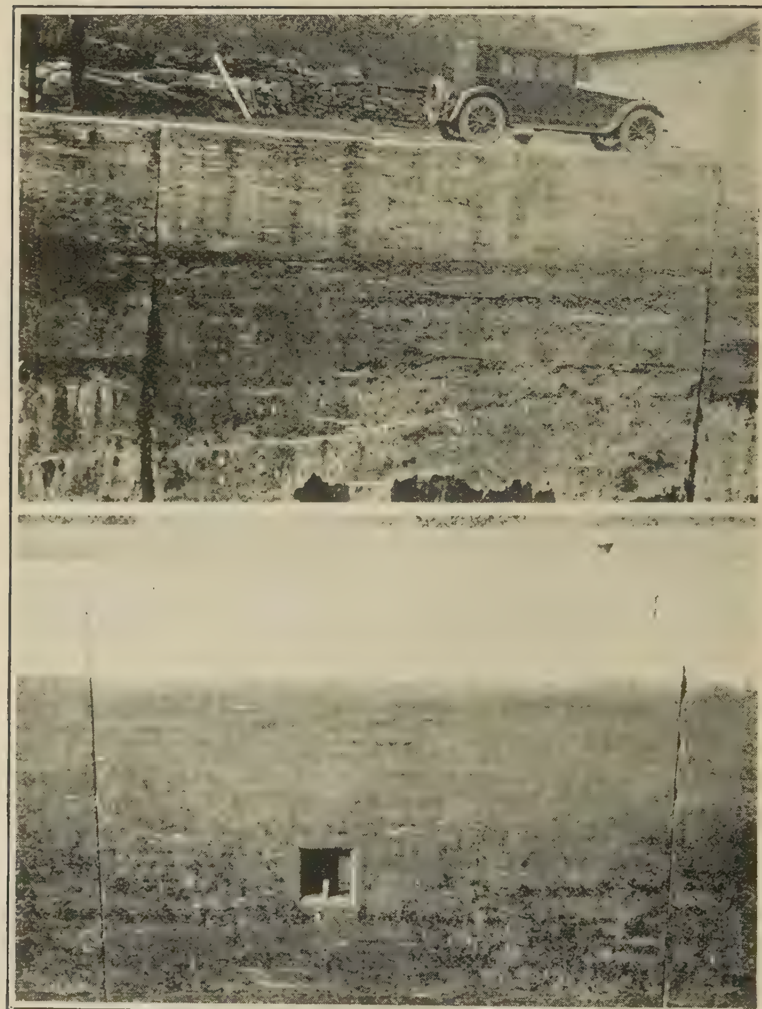


FIG. 17.—THESE TWO STRUCTURES WERE BUILT AT THE SAME TIME WITH THE SAME MATERIALS.

In the one above the inspection was not rigid and the work was carelessly done. In the other the concrete was placed under careful control. Note the fill planes, efflorescence and disintegration in the upper structure and the entire absence of disintegration in the lower.

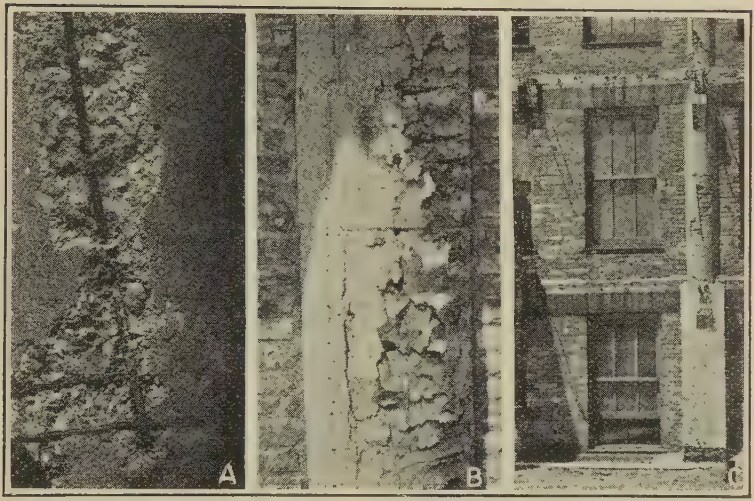


FIG. 18.—REINFORCEMENT CORROSION CAUSED BY WATER PENETRATING THE CONCRETE.

These pictures illustrate a very common condition of exposed concrete construction where moisture has penetrated and caused the corrosion of the reinforcement resulting in surface spalling of the concrete and further exposure and corrosion of the steel. The thickness of concrete protecting the steel was from ⅛ to ½ in. on the areas affected. The outer shell is quite porous and has the appearance of an over-wet mix. Where the steel is sufficiently covered with concrete the columns are in perfect condition. (a) shows spalling starting at a spacer bar and extending to the spirals. (b) shows general spalling over the entire exposed portion of the column where spirals are close to surface. (c) shows a general view with method of repair; the lower column has been restored and faced with brick. The records show a 1 : 2 : 4 concrete of limestone aggregate mixed very wet, placed in October, 1910. The affected columns are on exposed walls several stories above the street.

FIG. 19.—A CONCRETE BACKING OF A RUBBLE WALL, EXPOSED ONLY TO RAIN AND FROST FOR 9 YEARS.

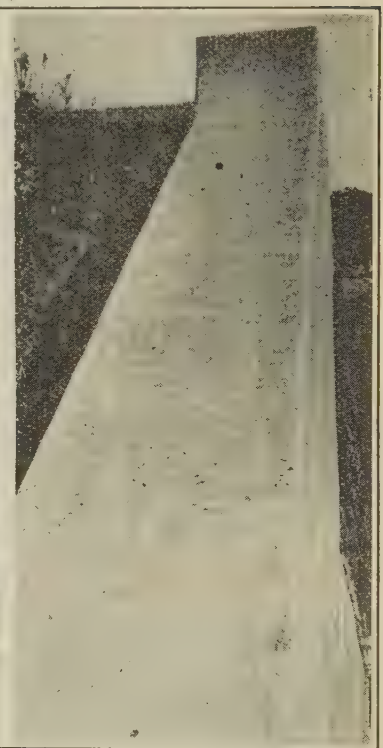


FIG. 20.—A WING WALL COPING AFTER 10 YEARS OF ABOUT THE SAME EXPOSURE AS THE WALL IN FIG. 19.

Another comparison illustrating the greater durability of concrete when properly placed. The wall in Fig. 19 has the appearance of a lean mix placed quite wet and over-spaded to give a smooth surface. Fig. 20 is a splendid example of concrete of correct consistency and mix.

DISCUSSION.

Russell S. Greenman (*By Letter*).—Because of my large opportunities for more than fifteen years of inspecting concrete in the making and "wearing" on big contracts and over considerable area, I would like to state that I am much impressed by the simplicity and the soundness of the conclusions reached in the Report of Committee E-6 on "Destructive Agents and Protective Treatments." Mr. Greenman.

It has long seemed to me that in the search for formulas and refinements by which to produce *good* concrete the simple conditions of density, imperviousness, and non-absorptivity have been too much overlooked. It is very satisfying then to read such a frank, sensible, and important review of the situation as is presented in the last paragraph of this report. Much more hardly need be said and yet I would call attention to three thoughts brought out by the report.

This report rightly lays large emphasis on the need of making concrete impervious. I would add to this that concrete may be dense and yet may absorb moisture enough to become susceptible to frost action. A dense, compact concrete can absorb moisture through the nature of its aggregates—especially the fine aggregate,—and some of these have been, and may be, the channels for frost disintegration. This characteristic should not be overlooked when designing mixtures or accepting materials.

The paragraph referring to "removal of laitance" contains this sentence: "The porous concrete immediately below the laitance still remains." This porous concrete is evident in the upper part of the "lift" or "construction unit" of too much of the concrete made. Over-wet mixtures are a cause, but another is that in the present day speed too often is emphasized at the expense of workmanship. The last part of the unit of poured concrete is simply struck off and is but seldom compacted. Density of the lower part of the unit is produced largely by the weight of the material above, but the upper part has not the benefit of such weight. Therefore greater care and better labor should be used more.

Just a concluding thought. The deposit on the face of concrete is spoken of as "efflorescence." Technically, this may be correct. But is not that material found in conditions such as is referred to in connection with the illustration of "Dam A" much more than efflorescence; is it not more of an "incrustation" since it may have considerable thickness in the scale and may even be not in direct contact with the surface?

C. A. P. Turner.—In one of these photographs the aggregate was Minneapolis blue limestone if I am not mistaken. I have never seen any concrete exposed to the climate of Minnesota that would stand up when Mr. Turner.

made of that material. My idea of getting permanent concrete is first see that the aggregate is of permanent material—one that cannot be disintegrated by frost as that kind of limestone disintegrates.

Work put in with the same consistency or cement-water ratio using clean washed gravel, has stood up whereas this limestone concrete has disintegrated.

We have examples of overhead bridges exposed to the locomotive exhaust where there was no disintegration when washed gravel was used in the concrete and on the other hand where this limestone was used 2½ in. disintegrated exposing the steel underlying surface and similar clearance heights.

Mr. McMillan. F. R. McMILLAN.—I call attention to Fig. 18A, in which only one-third is disintegrated. It is all made of the same limestone. Just why all of it has not disintegrated in fifteen years if the limestone is at fault, I will let Mr. Turner explain.

Mr. Turner. MR. TURNER.—Limestone of that kind has layers which the quarrymen call soapstone, merely hardened sea bottom slime. Some of that is distributed through various layers of this stone. There may be occasionally a few feet of fairly hard material. That is the difference between stone from the same quarry which will stand for a period of years when built in a wall and stone which disintegrates in a short time. The same difference will be found in the same stone from the same quarry when crushed from different layers having these divergent characteristics.

Prof. Hatt. PROF. W. K. HATT.—I think we should make some distinction between these various kinds of limestone, as Mr. Turner has indicated, because the oolitic limestone, as used for building purposes, will certainly stand exposure. There are certain bastard limestones that lie above the oolitic which contain clay seams.

Mr. McMillan. MR. McMILLAN.—I know that such stone is undesirable, but it is very curious how the soapstone gets between the rods and the surfaces where the rods are near the surface, and more where they are forward. This failure is due to the nearness of the rod not to the soapstone or anything else. Wherever there is failure the rods are 1 in. from the surface; where the material is good the rods are back from the surface.

Mr. Merriman. THADDEUS MERRIMAN.—As long as the concrete remains alkaline the steel will not rust. The alkalinity of the concrete disappears by reason of the combination of the calcium hydroxide it contains with carbonic acid from the atmosphere. It is very easy to trace this carbonation process from the surface toward the center of the mass and when it has extended far enough to reach the steel, rusting at once begins.

Mr. Johnson NATHAN C. JOHNSON (*By letter*).—My original comment made at the time of presentation of this report at the meeting at Chicago is no longer germane inasmuch as the report has been modified.

Mr. Johnson.

A careful perusal of this report, however, seems to require as comment that it would be undesirable to have this report stand as representing the present state of knowledge of the art of making concrete as it is embodied in the many examples at hand of how *not* to make concrete.

The report of this committee is interesting but elementary. The litany of ascribing disintegrations to "excess water, improper mixing and improper placing" is soothing but does not lead us to any useful conclusions.

The photographs shown might even be said not to represent disintegrations but merely to represent the common result produced in the art. The only disintegration that the writer has noticed in any of the structures cited by the committee (and he is personally familiar with all but two) are simply a falling away of the obscuring outer skin with passage of time and a disclosure of the ugly facts underneath.

It would have been much more to the purpose if the committee had considered from an expert viewpoint the disintegrations of stadiums and other structures where muck streaks and the like are not the predominating effect. There is a vast amount of information already at hand which bears upon true disintegrations; and there is also a vast amount of knowledge which is as yet unacquired or at least unclassified.

It may be said as a demonstrable fact that every disintegration and its causes can be accurately determined by expert examination and that there is nothing which more truly leads to a knowledge of how to make enduring concrete than a proper study of disintegrations.

At the present time this knowledge is not wide-spread. Because men are skilled in artistic design does not mean that they are qualified either to construct for permanence or to state the cause of disintegration. Because certain men are slide-rule wizards does not mean that the structure which they design according to certain assumed formulas will have a useful or even reasonable life and endurance. Because certain other men are skilled in handling a gang or in laying out a concrete plant or in cutting costs does not mean that they have other than rudimentary knowledge as to what is necessary to prevent disintegration and to insure permanence.

The burden under which the art at present labors is very largely the assumption of universal understanding by each one of these several groups; and the conferring on a vast structure of a sort of Papal blessing through virtue of a specification (which often is atrocious in itself) and by a few laboratory tests to confirm one formula or another as to strength.

It would be a very great benefit to the art if the committee would reassemble and consider disintegrations in their true significance and publish complete data on this subject.

Report of Committee E-4 Fire Resistance of Concrete.

Committee E-4 on Fire Resistance of Concrete was authorized by the Board of Direction at a meeting held in April, 1923. The purpose of the committee was the "study and review of test data and the results of fires on, and the fire resistance of, various types of concrete and make a report as early as possible on the comparative fire resistance of these various types of concretes with special reference to the kind of aggregate." The committee was organized in November, 1923, and assignment of subjects to the members made. As the work of the committee had barely got under way before time for the 1924 convention no report was made other than an outline of the proposed work and a list of the assignments was sent to the secretary.

A further assignment of work for the committee was made in June, 1924, with the request that the committee report "what, if any, action should be taken on the matter of adoption of the 1918 Proposed Standard Specifications for Fire Tests of Materials and Construction."

This specification is now undergoing a thorough revision in preparation for advancement as an American Engineering Standard. The sponsors for the work are the American Society for Testing Materials, the fire-protection group of the American Engineering Standards Committee, and the United States Bureau of Standards. The membership of the committee working on these revisions includes all those participating in the original work with the exception of the Engineering Institute of Canada and has been extended to include the Canadian Engineering Standards Association (through a liason member), the Office of the Supervising Architect, U. S. Treasury Department, the Building Officials Conference, Prof. Albin H. Beyer, of Columbia University, an independent specialist in this kind of testing, and eight of the large organizations of industries directly affected. In addition to the revision of the previous specification, it is proposed to extend the standards to include specifications for fire tests of materials and structural units which constitute permanent integral parts of a finished building.

The work of this committee, under the able direction of Ira H. Woolson as chairman, is progressing very well and it is quite possible that it will go to the sponsors this year and, if acceptable, will doubtless be sent to the main committee for adoption.

In view of the foregoing your committee, E-4, recommends that no action be taken at this time toward the adoption of the 1918 Proposed Standard Specifications for Fire Tests of Materials and Construction as standard.

Through assignment of various subjects to the several members of the committee studies of test data and reports of fires in concrete buildings have been made and certain suggestions on the design and construction of reinforced-concrete structures so as to provide greater resistance to fires were obtained. No meeting of the committee has been held. It had been the hope of the chairman, however, that when the material from the different assignments was ready, to call a meeting of the committee to review the whole report in detail and put the information of the collected studies in a form more useful to the Institute membership. The mass of data and reports to be reviewed, the variations in test methods and the manner of reporting tests, the seemingly contradictory results of tests in some cases and the inability to make direct comparisons have made the work of correlation very difficult. No review of reports on the fire resistance of building units (blocks, etc.), plastered constructions nor gunite walls is included, as these have been reported by others.

Probably the most valuable contribution to the study of the effects of fire on concretes in which the various aggregates are used was the series of tests of the special commission formed by the British Fire Prevention Committee. And next in importance the tests of the Bureau of Standards at Pittsburgh and those at the Underwriters Laboratories in Chicago made jointly by the Underwriters Laboratories, The Factory Mutual Insurance companies and the Bureau of Standards. These tests gave results which correspond with the conclusions drawn from Prof. Woolson's tests which were reported to the American Society for Testing Materials in 1905, 1906 and 1907 and will be reviewed very briefly. There is appended also a résumé drawn from our studies and a few admonitions concerning details for securing fire resistance in reinforced-concrete buildings. Only a few of the reports of the effects of fires in buildings have been studied carefully. Those from which we have been able to get the most valuable information on the effects of fire on the concrete are:

Report of Committee on Edison Fire, *Proceedings*, American Concrete Institute, 1915, p. 585.

Fire at the Venesta Works, Silvertown, London, British Fire Prevention Committee Red Book No. 211, 1917.

Fire in a Reinforced-Concrete Warehouse at Far Rockaway, New York, by Ira H. Woolson, published by the National Board of Fire Underwriters, New York, and by the British Fire Prevention Committee, London, as Red Book No. 214.

The Fire on the Quaker Oats Co.'s premises, at Petersboro, Ontario. British Fire Prevention Committee's Red Book No. 225, 1918.

Lessons in Fire-Resistance from the Frankford Fire—W. A. Hull, *Proceedings*, American Concrete Institute, 1921, p. 205.

Report of Committee on Far Rockaway Fire, *Proceedings*, American Concrete Institute, Vol. XVII, 1921, p. 368. (This also gives a brief report of the fire in the Barrett Manufacturing Co.'s plant at Frankford, Pa.

In general, it may be stated that these reports of fires confirm fully some of the conclusions drawn from the reports of tests mentioned above, particularly with respect to the susceptibility of certain aggregates to spalling in fires.

Another phase of the problem of fire resistance is the determination of the probable intensity and duration of fires in the various building occupancies and under different kinds of exposure conditions. Careful observation of fires in buildings should give some light on this problem. There is need also of investigation of the subject by means of such tests as can be made in houses and buildings representative of the various classes of construction to be considered. From the literature of the subject not much information can be had, but such as there is, is of great value. It is particularly important to know what are the effects of increasing areas between fire walls or partitions, and what are the effects of fires burning in successive stories of fire-resistive buildings. There are also to be considered conflagration conditions, which appear to be the most severe of all in their destructive effects on buildings.

Little information as to the effect of thickness of members on the fire resistance of reinforced-concrete structures has been found. Some experiments have been made with columns[1] and floor slabs[2] of different thicknesses and kind. It has also been observed in fires in buildings that there is some tendency for structures having thin floor slabs to thick beams to suffer from unequal expansion effects. Nothing of a quantitative nature is known of this either. Both of these subjects need further study.

There is appended to this report a brief bibliography of "the fire resistance of concrete." In presenting this we wish to express our appreciation of the assistance of Prof. Duff A. Abrams, of the Structural Materials Research Laboratory, Lewis Institute, Chicago. From his "Bibliography on Effect of Fire on Concrete" a very large number of the references listed herein were taken. Other references, from various sources, include a number which relate particularly to building stone. These have been included as of possible interest in the consideration of the different kinds of aggregate. There are also numerous references to abstracts and duplicates of articles appearing elsewhere in the list. However, these have been included[1] because some libraries might have some, but not all, of the books or periodicals in which a subject is treated, and[2] because occasional comments, and references, sometimes appearing in the briefer articles, might be of interest.

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[1]Tests of Concrete Columns at the Pittsburgh Laboratory, of the U. S. Bureau of Standards.

[2]Tests conducted by the Special Commission formed by the British Fire Prevention Committee, see foot note on page.

The committee plans, with the consent of the Institute, to proceed with gathering such additional material as it can concerning the fire resistance of concrete and will try to present it in a form to make it more readily available to those interested. This report represents the progress of the work so far.

The committee consists of five members, all of whom have approved this report.

NOLAN D. MITCHELL, *Chairman*,
J. R. DWYER, *Secretary*.

TESTS OF CONCRETE BY THE BRITISH FIRE PREVENTION COMMITTEE.*

This series of tests, which started in October, 1917, and lasted until September, 1919, was the most comprehensive ever undertaken. The primary object was to study the effect of fire on concrete made with different aggregates. It is unfortunate that it was discontinued before the completion of the program. The site of the testing station was acquired for other purposes and the apparatus had to be dismantled.

Tests were made of both plain and reinforced-concrete slabs having natural and (or) artificial aggregates for fire resistance, resistance to erosion from hose streams and for heat conductivity of the materials. Auxiliary tests were made for compressive, bending and shearing strengths of the various concretes tested. Comprehensive studies of the aggregates and concretes were made.

Ninety-two plain concrete slabs 10 ft. 8 in. long by 3 ft. 6 in. wide and 5½ in. thick, exposing 10 ft. by 2 ft. 9 in. of the soffit, were fire tested, while under superimposed loads. Of these 49 had natural aggregates, the others artificial aggregates. The natural aggregates included six kinds of gravel from seven sources, seven kinds of sandstone, four limestones, and eleven igneous rocks, five of which were commonly classed as granites. The artificial aggregates included five kinds of brick, a burnt clay, blast-furnace slag from two sources, coke breeze and pan breeze from two sources each and clinkers from three (two from bituminous coals, one from anthracite).

The 67 slabs of reinforced concrete were 11 ft. 6 in. square with exposed soffits 10 ft. square. The two-way reinforcement of ⅜-in. plain round rods, alternately of lengths 11 ft. 3 in. and 9 ft. 3 in., with fish-tailed ends, was placed in various slabs at ½ in., 1 in., 1½ in., and 2 in. from the bottom of the slabs. The effective depth of the slabs varied from 3⅛ to 3¾ in., the greater depths being required for the concretes of lower strength. The computations for the strength of the slabs were made on the basis of the French rule in which the bending moment in

*British Fire Prevention Committee Red Books: Nos. 101, 212, 213, 216, 217, 218, 219, 224, 228, 237, 243, 249, on plain concrete slabs; Nos. 221, 222, 223, 226, 227, 229, 231, 232, 234, 238, 239, 242, 244, 246, 248 and 250, on reinforced concrete slabs; Nos. 251 and 252, on conductivity of heat through slabs; No. 256, geological notes and descriptions of aggregates; No. 257, mechanical tests of concretes; and No. 258, synopsis of the tests.

each direction is considered to be equal to $\frac{Wl}{24}$. Thirty-five of these slabs had natural aggregates: 16 of gravels (two kinds), 4 of sandstones, 3 of limestones, and 12 of igneous rock. Of the twenty-eight slabs having artificial aggregates 10 had broken brick, 4 crushed burnt clay, 6 coke breeze, 4 pan breeze, 3 clinker and 1 blast-furnace slag. The total thickness of the reinforced slabs varied from 4 in. to 5½ in.

The fire test period for 44 of the plain concrete slabs was three hours followed by a two-minute application of water from a ⅝-in. nozzle (pressure nearly 50 lb. per square inch). Twelve were similarly exposed to fire without the subsequent application of water. The superimposed loads were 224 lb. per square foot. Twenty-two slabs were loaded to 280

RESULTS OF CONDUCTIVITY TESTS BY THE BRITISH FIRE PREVENTION COMMITTEE.

	Number of Sample Tested	Kind of Coarse Aggregate	Kind of Fine Aggregate	Mix	Order of Merit for 1-in. Protection[1]	Order of Merit for 2-in. Protection[2]
Natural Aggregates	5	Limestone	Sand	1:2:4	100	100
	4	Whinstone Traprock	Sand	1:2:4	89	95
	2	Basalt	Sand	1:2:4	84	99
	6	Sandstone	Sand	1:2:4	83	96
	3	Granite	Sand	1:2:4	69	90
	2	Irish Pit Pebble	Sand	1:2:4	67	66
	3	Gravel	Sand	1:2:4	66	87
Artificial Aggregates	1	Pan Breeze	Fine Pan Breeze	1:2:4	91	88
	2	Burnt Gault Clay	Sand	1:2:4	88	121
	3	Slag	Sand	1:2:4	81	81
	2	None	Fine Coke Breeze	1:5	78	108
	4	Brick	Sand	1:2:4	73	96
	3	Coke Breeze	Sand	1:2:4	72	82
	3	Clinker	Sand	1:2:4	61	86
	1	Coke Breeze	Fine Coke Breeze	1:2:4	29	53

[1] Based on the time required for concrete to attain 1000 deg. F. at one inch from soffit of slab.
[2] Based on the relative temperatures attained at the end of 4 hours. In each case limestone concrete considered as 100 as an arbitrary point on the scale.

lb. per square foot and subjected to four hours' fire and five minutes' application of water. Fourteen other slabs had protective coatings of plaster and (or) concrete applied to the soffit. Most of the slabs were of 1 : 2 : 4 mix; there were, however, several in which richer mixes were used and in some instances fine aggregates other than sand were employed.

The gravels, sandstones, granites and other highly siliceous aggregates were unsatisfactory, while the fine-grained igneous rocks, classed as basalt, dolerite and trachyte gave satisfactory results. The limestones were better than the siliceous aggregates, the fine-grained stone being distinctly better than the coarse-grained.

Coarse aggregates of clean broken bricks and burnt clays gave the most satisfactory results from strength and resistance to fire. Blast-furnace slag appeared to be almost as good. Pan breeze and clinker were fairly satisfactory as aggregates for plain concrete slabs. In some cases

coke breeze used as fine aggregate gave better results than sand in comparable slabs.

Not all aggregates suitable for plain concrete slabs give good results in reinforced concrete, usually, however, aggregates which are unsuited for one are not suitable for the other. This has proven to be the case with the gravels, sandstones, and the coarse-grained and highly siliceous rocks. Those aggregates which gave the best results in plain concrete have proven to be the most satisfactory ones in the reinforced-concrete slabs. Pan breeze and coke breeze which gave fairly satisfactory results in plain concrete slabs did not give satisfactory results in the reinforced slabs.

Siliceous gravel in equal proportions with basalt for the coarse aggregate in 1 : 2 : 4 mixtures did not give satisfactory results in either plain or reinforced concrete. Nor were the siliceous gravel concretes with 1-in. protective coatings of coke breeze, pan breeze, or broken brick concretes on the soffit satisfactory in either the plain or reinforced slabs, except in the case of thoroughly dried, year-old slabs. One case of 2-in. protective coating gave satisfactory results with a plain slab. One-half inch of gravel concrete plus 1 in. protective coatings of fine coke or pan breeze mortars as protection to the steel in reinforced-concrete slabs was not sufficient.

In parallel with the tests of plain and reinforced-concrete slabs the heat conductivity of the various concretes was measured. In 22 of 30 tests the temperature at 1 in. from the soffit was 1,200 deg. F. or higher at four hours. The coarse aggregates in the slabs which did not attain to 1,200 deg. F. were slag, limestone (2), basalt, andesite, coke breeze, broken brick and dolerite (whinstone). The highest temperatures at points near the top of the slabs were attained with concretes having as coarse aggregates siliceous gravel (2), calcareous gravel, coke breeze and quartzite.

COLUMN TESTS AT THE PITTSBURGH LABORATORIES OF THE BUREAU OF STANDARDS.*

The following is a summary of the results of the fire tests of concrete and reinforced-concrete columns made at the Pittsburgh laboratories of the Bureau of Standards by W. A. Hull and reported by him in the *Proceedings* of the American Concrete Institute. The investigation was designed to secure information as to the effects of the kind of aggregate, the type of reinforcement and the shape of the cross-section on the fire resistance and the strength of the column at high temperatures.

The fire tests were made on 62 columns subjected to normal working loads. Compression tests were made on 16 comparable columns which

*Preliminary Reports:
Fire Tests of Concrete Columns, W. A. Hull, *Proceedings*, American Concrete Institute, Vol. XIV, 1918; Vol. XV, 1919; Vol. XVI, 1920.
Final Report:
Bureau of Standards Technologic Paper No. 272, Fire Resistance of Concrete Columns, by W. A. Hull and S. H. Ingverg, physicists.

were not subjected to fire. Those columns which did not fail during the four-hour test were subjected at once to additional loads to determine their strength while hot. Three columns were of plain concrete without reinforcement, 23 had 2 per cent of vertical reinforcement with only a nominal amount of lateral reinforcement in the form of ¼-in. wire ties spaced 12 in. apart, and the others had 2 per cent of vertical reinforcement of bars and 1 per cent lateral reinforcement of 5/16-in. steel wire in the form of spiral hooping. Thirteen of the columns were 16 in. square, the others round with diameters of 12, 18 and 20 in.

Since it had been indicated in the previous fire tests that the kind of aggregate had a decided influence on the fire resistive properties of concrete a wide range in the mineral composition of aggregates was introduced. Most of the columns had protective coverings of concrete cast monolithic with the column, some, however, had other kinds of protective covering. The coverings were made 1½ in. or 2½ in. in thickness. Three had light-weight expanded metal; others wire mesh placed near the surface to prevent the covering materials from being dislodged in cases of cracking and spalling. Some of the coverings were plastered on metal lath. The temperature in the gas-fired furnace was increased as nearly in accordance with a predetermined time-temperature curve as possible.

The tests gave widely different results due mainly to the differences in the mineral composition of the aggregates. Aggregates with high quartz, chert or granite content are likely to induce spalling or serious cracking of the concrete when subjected to fire of moderate intensity and duration. On the other hand, concretes made with calcareous aggregates, such as limestone or calcareous gravel, suffer few visible effects even when exposed to very severe fires of four hours' duration. Concretes made with trap rock or blast-furnace slag give results intermediate between these.

The poor showing of the concrete columns made with siliceous aggregates when subjected to fire is due mainly to the expansion characteristic of quartz and other forms of silica and minerals containing them. They spalled and cracked very badly, exposing the reinforcement in most cases to the high temperatures of the furnace. Increasing the thickness of the covering to 2½ in. did not give suitable protection. Siliceous materials are so important economically and since they constitute the only available aggregate in many localities experiments were made to determine whether concretes made with them could be protected against the serious fire effects here noted. Some columns were protected by the application of plaster on metal lath, others, by a special mixture of asbestos and other fire-resistive materials bound on with wire netting, and some by placing a light-weight expanded metal sheet inside the forms before casting the columns. The first two methods are applicable to structures already erected, the latter only to those in process of construction. The light-weight expanded metal, when placed in 2½-in. protective concretes made of siliceous gravel and at the proper distance from the surface effected a great improvement. The columns so made withstood fires of four hours' duration while subjected

to normal loads, and at the end, loads from two to over four times as great before failing. The effect of adding secondary reinforcement to the 1½-in. protection is to increase the fire resistance period for columns of 200-sq. in. sections from 2½ hours to 3½ hours and when added to the 2½-in. protection, to increase the fire resistance period from 3 hours to 6 hours. Columns made with trap rock or blast-furnace slag concrete suffered less damage from the fire and consequently had greater ability to come through the tests without failure. Columns made with concrete having limestone or calcareous gravel aggregates made quite the best showing of all columns having concrete protection. They did not crack or spall extensively, nor did the dehydration of the concrete reach to as great a depth. The limestone near the surface was calcined. Concretes made with these aggregates had better heat insulating qualities than the others.

Such columns with the proper amount of vertical and lateral reinforcement protected by 1½ in. of concrete outside of the steel resist test fires comparable to the most severe fire conditions that may reasonably be expected with almost any building occupancy. The shape of the column section whether round or square was, so far as could be observed, of minor importance with respect to fire resistance. The type of reinforcement, however, has some small influence. Columns without reinforcement, when not subjected to bending or excessive stresses, may be expected to resist fire as well as reinforced columns. With spirally-hooped reinforcement there is some tendency to separation of the protective covering from the column along the plane of the spiral when siliceous aggregates are used, and round columns of such concrete with vertical reinforcement and lateral ties are slightly more resistant to fires than square ones. These effects on 16-in. diameter round columns of 1 : 2 : 4 in. of monolithic protective covering outside of the vertical reinforcement when expressed in resistance time are as follows:

When lateral ties are used the fire resistance period is 3 hours; and 2½ hours when spiral hooping is introduced. Changing the section from round to a square one of the same gross area also reduces the resistance period to 2½ hours. The embedment of light-weight expanded metal or wire mesh in the protective covering to prevent spalled portions from falling away increases the resistance periods of all three types to 3½ hours.

TESTS AT THE UNDERWRITERS LABORATORIES, CHICAGO.*

Fire tests were made of 106 columns. All were 15 ft. 8 in. long and had 12 ft. exposed to the fire. The testing furnace, partly open, is shown

*Technologic Papers of the Bureau of Standards No. 184, Fire Tests of Building Columns, by S. H. Ingberg, Physicist; H. K. Griffin, Associate Physicist, Bureau of Standards; W. C. Robinson, Vice-President, Underwriters Laboratories; R. E. Wilson, Engineer, Associated Factory Mutual Fire Insurance Companies.
Fire Tests of Building Columns by Associated Factory Mutual Fire Insurance Companies, The National Board of Fire Underwriters and the Bureau of Standards, published by the Associated Factory Mutual Insurance Companies, Boston, and the Underwriters Laboratories, Chicago.

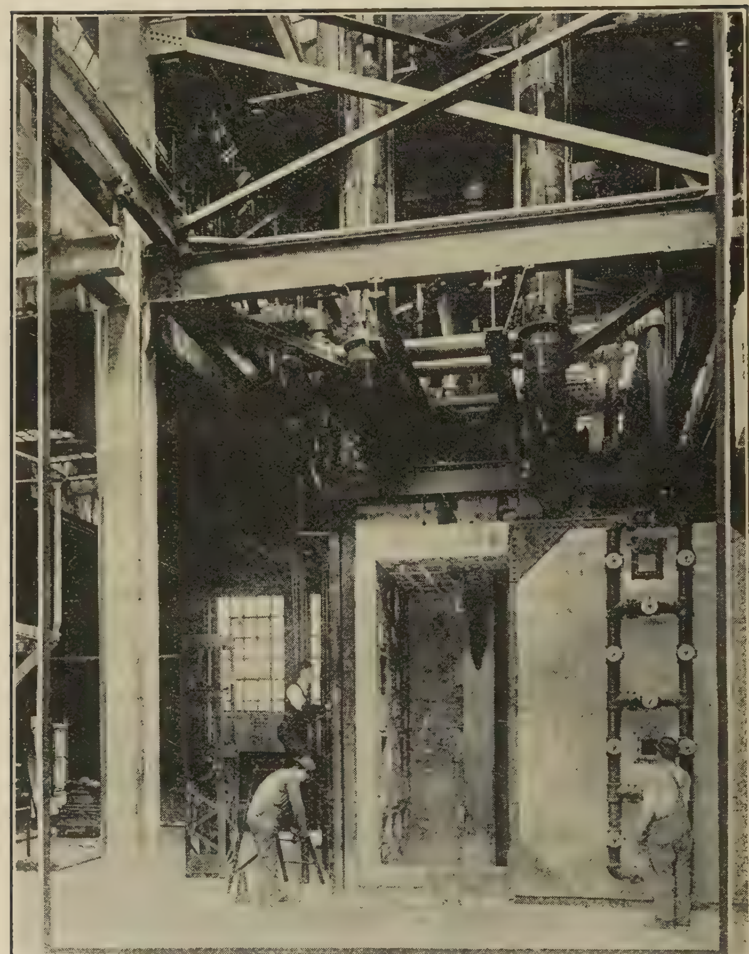

FIG. 1.—VIEW OF COLUMN TESTING EQUIPMENT USED IN MAKING FIRE TESTS ON BUILDING COLUMNS AT UNDERWRITERS' LABORATORIES. FURNACE WITH SAMPLE INSTALLED AND ABOVE IT THE HYDRAULIC CYLINDER FOR LOADING.

in Fig. 1. Loads were applied to the columns by means of the hydraulic cylinder mounted above the furnace. There were 91 fire-endurance tests, and 15 for fire and water effects. Included in the fire-endurance tests were unprotected and partly protected columns of timber, structural steel, steel

FIG. 2.—SQUARE VERTICALLY REINFORCED COLUMN: 1: 2: 4 CHICAGO LIMESTONE CONCRETE. AFTER 8 HOURS 40 MIN. EXPOSURE TO STANDARD FIRE CONDITIONS AND 3 TIMES ITS NORMAL WORKING LOAD.

pipe and cast iron; and protected columns of timber, structural steel, cast iron and reinforced concrete.

The covering materials used to protect the columns were: Cement plaster on metal lath, hollow clay tile, gypsum block, brick, and concrete. The concrete for the protection of the structural steel and cast-iron columns was in most cases of 1 : 2 : 4 mix. There were, however, a few examples of other mixtures, 1 : 2 : 5 cinder concrete and 1 : 3 : 5 stone concrete. The

thickness of the protection was either 2 or 4 in. Typical sections of columns having concrete protective coverings are shown in Fig. 2. The aggregates used were (1) Plum Island (Mass.) sand and Rockport granite; (2) Long Island (N. Y.) sand and hard coal cinders; (3) Long Island sand and New York trap rock (diabase); (4) Pelee Island (Ont.) sand and Cleveland sandstone; (5) Meramec River (Mo.) sand and Meramec River gravel; (6) Fox River (Ill.) sand and Joliet (Ill.) gravel; and (7) Fox River sand and Chicago limestone. The combinations of aggregates 3 and 7 only were used in the reinforced-concrete columns for fire tests. All the combinations of aggregates were used in the fire and water tests by making a 3-ft. section of the column with one kind, the next 3 ft. of another, etc.

For both the concrete protections to the steel columns and the reinforced-concrete columns square sections with slight chamfers on the corners and round sections were made to determine whether the shape of the section of the column had any very great effect on its fire-resistive qualities. The influence from this cause had little effect, so far as could be observed, or at least was so small compared with the effects from other causes that it was not possible to draw any definite conclusions. The mechanical properties of the concretes varied widely, the strength from 1,560 to 2,100 lb. per square inch at 28 days, and the modulus of elasticity from about 1,000,000 to over 4,000,000 lb. per square inch. The fire resistance, it seems, was not seriously affected by these variations in compressive strength.

The mineral composition of the aggregates had the most influence on the fire-resistive properties of the concrete. Not only were some of the concretes distinctly better than others but showed greater recovery of strength after exposure to fire.

With regard to the variations of the fire resistance of the concretes with respect to difference in aggregates used, it might be well to quote from Technologic Paper No. 184, p. 179, "With a given thickness or size of covering the main cause of variation in results was the difference in the fire resisting properties of concrete made with different aggregates.

In this particular the concrete can be placed in three groups. That giving the most unfavorable results was the concrete made with the Meramec River sand and gravel, a number of large cracks forming early in the tests followed by spalling of large and small pieces of concrete not held by the ties. This sand and gravel consists almost wholly of quartz and chert grains and pebbles, the gravel having a particularly high chert content. Both minerals are forms of silica (SiO_2) the quartz being crystalline and anhydrous, and the chert amorphous, with a variable amount of water in chemical combination. On being heated part of the combined water in chert is liberated, and the consequent vaporization disrupts the pebbles. Other causes of disruption of concrete made with siliceous aggregates are abrupt volume changes, points of which are known to exist for chert as as low as 210 deg. C. (410 deg. F.). Quartz has a decided point of abrupt volume change at 573 deg. C (1063 deg. F.), where it is transformed into

Beta quartz and later into the mineral tridymite, the change extending over a considerable temperature range when the heating is rapid. Liquid inclusions contained in small cavities formed when the rock crystallized from the molten condition, may be the cause of some of the cracking incident with fire exposure.

The middle group includes concrete made with trap rock, granite, sandstone, and hard coal cinder. In tests with trap rock and cinder concrete a small amount of cracking developed during the last part of the fire period, but no spalling of note occurred before failure. In the granite

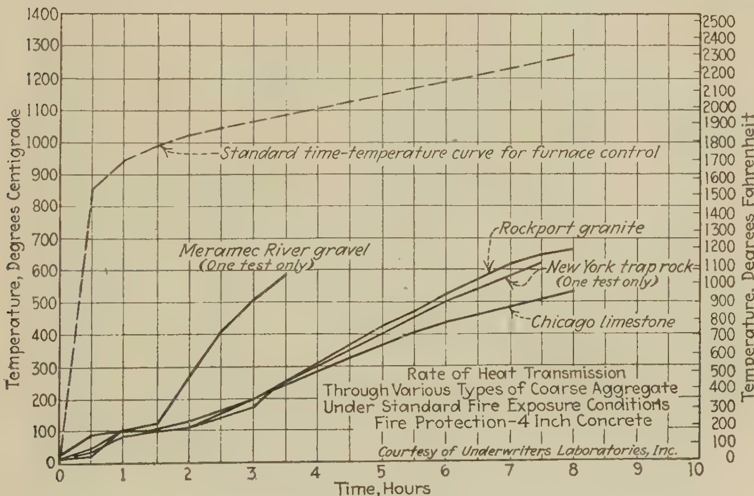

FIG. 3.—RATE OF HEAT TRANSMISSION THROUGH VARIOUS AGGREGATE: 4-IN. CONCRETE.

concrete protections cracking and spalling of the corners outside of the wire ties began in the first 30-min. period and continued during the next hour, after which there was little apparent change before failure. The spalling exposed portions of the flange edges, which to some extent hastened the failure. The average time to failure in tests with sandstone-concrete protections was intermediate between those with trap rock and those with cinder concrete. The cracking of sandstone concrete after a short fire exposure can be ascribed mainly to the abrupt volume change of the constituent quartz grains as noted above.

Fusion of the trap rock concrete occurred where the tests extended beyond 7 hours, the concrete being affected to a depth of about 1½ in. Flowing of concrete due to fusion, while not general, occasionally formed pockets up to a 2-in. depth. Incipient fusion to about the same depth

occurred in the 4-in. granite-concrete protections, although no actual flowing of concrete took place.

The third group comprises protections of Chicago-limestone concrete and Joliet-gravel concrete. The composition of this gravel is similar to that of Chicago limestone, and the fire resisting properties of the concrete made with each compare quite closely. Very little cracking resulted on exposure to fire and their heat-insulating value was increased by the change of the calcium and magnesium carbonates to the corresponding oxides. This process retarded the flow of heat through the region of change and left material of good insulating properties. Immediately after

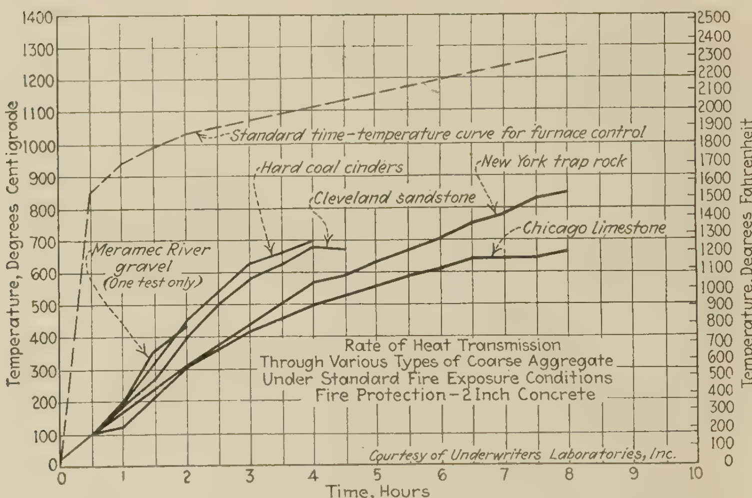

FIG. 4.—RATE OF HEAT TRANSMISSION THROUGH VARIOUS AGGREGATE: 2-IN. CONCRETE.

the test the surface of the concrete was firm, but after a few weeks' exposure the hydration of the oxides caused slaking and crumbling of the calcined materials.

Figs. 3 and 4 show the rise of temperatures at depths of 2 in. and 4 in. from the surface of columns. These rates of rise of temperature are in a measure an indication of the heat transmission through the concrete made with the various aggregates. Meramec-River-gravel and hard-coal-cinder concrete fall in the class showing the greatest rise of temperature in a given time, while limestone concrete shows the least.

The features of greatest interest in connection with the tests are the useful limits of the protective coverings when subjected to fire and the value of the column afterwards. Ratings in fire resistance made after all

phases of the tests had been carefully considered are given in the following table:

FIRE RESISTANCE PERIODS DERIVED FROM THE TEST RESULTS.

Type of Column	Protection: Material	Protection: Details	Minimum Area of Solid Materials, sq. in.	Nominal Thickness of Protection, in.	Fire Resistance Period, hours
Structural Steel....	Concrete; sil. gravel aggregate.	Mixture 1:6 tied with steel ties or wire mesh equivalent to not less than No. 5 (B. & S.) wire on 8 in. pitch.	100	2	1
Structural Steel....	do....................	do....................	200	4	2½
Structural Steel....	Concrete; granite sandstone or hard coal cinder aggregate.	Mixture 1:6, concrete tied as above.	100	3	2½
Structural Steel....	do....................	do....................	140	3	3½
Structural Steel....	do....................	do....................	200	4	5
Structural Steel....	Concrete; trap rock aggregate.	Mixture 1:6 tied as above.	100	2	3
Structural Steel....	do....................	do....................	140	3	4
Structural Steel....	do....................	do....................	200	4	5
Structural Steel....	Concrete; limestone or calcareous gravel aggregate.	Mixture 1:6. Concrete tied as above.	100	2	4
Structural Steel....	do....................	do....................	140	3	
Structural Steel....	do....................	do....................	200	4	8
Reinforced Concrete	Limestone or calcareous gravel concrete.	Mixture 1:6. Concrete reinforced with vertical bars and lateral ties or hooping.	220	2	8
Reinforced Concrete	Trap rock concrete.....	do....................	220	2	5

The condition of the 1 : 2 : 4 limestone reinforced-concrete column No. 70 after the fire test was as follows:

This column was 16 in. square. The reinforcement consisted of four 1-in. square bars with ¼-in. diameter wire hoops spaced 12 in. c. to c. The area within the hoops was 12 by 12 in., the outer 2 in. of the column serving as the protective covering. The working load sustained throughout the 8-hour fire test was 101,000 lb. After eight hours, with the fire continued, the load was increased until at 8 hours 40½ min. the column failed, under a load of 294,000 lb. The factor of safety of this column at the end of the 8-hour test was approximately three. After the failure the concrete near the outside was found to be calcined, but the surface was hard due to the incipient fusion of the sand.

A column identical in all details except that the aggregates were Long Island sand and New York trap rock failed under the same working load (101,000 lb.) at 7 hours 22¾ min. Comparative tests of one pair of

round columns with six 1-in. square bars placed vertically and ¼-in. diameter wire hoops, and another pair with six ¾-in. square bars and ¼-in. spiral hooping with 1½-in. pitch gave results closely comparable to those of the square columns, the trap-rock-concrete column with the spiral hooping withstood the 8-hour test and a load 25 per cent in excess of the working load at its conclusion.

In the fire and water tests of reinforced-concrete columns the 1-hour fire affected the limestone and trap rock concretes very little, while the siliceous gravel concrete cracked and spalled, exposing portions of the reinforcement in the spirally hooped column. The water application carried away the concrete previously damaged by fire and some adjacent portions. The vertical bars of the square column were quite generally exposed, and bars and spiral hooping of the round columns in the portion made with the siliceous gravel concrete. After the fire and water tests the columns were loaded to about four times the working load before failure occurred. The condition of the columns was not such that they would be subject to early failure in the case of a recurring fire.

RESUMÉ.

The lessons that may be drawn from the studies so far made by the committee are that the fire resistance of concretes depends to a great extent on the kinds of aggregates used. Aggregates, such as the siliceous gravels used in the tests reviewed, result in concretes which are likely to spall rather quickly when exposed to fires. Small percentages of chert or other highly siliceous aggregates mixed with aggregates which do not spall may still cause serious cracking and spalling. Sandstones and granites vary somewhat in affecting the fire resistance of concretes in which they are used as aggregates, but usually the results are slightly better than with the siliceous gravels. Both have the tendency to crack and spall. Hard-coal-cinder concrete does not show this tendency but transmits heat more readily, therefore, does not give longer protection to the steel and structural concrete. Concrete made from blast furnace slag gives results in fire tests about equal to those of trap rock concrete. Both are decidedly better than concretes having the highly siliceous aggregates. In nearly all tests limestone has been shown to be superior to all the other natural aggregates in its fire resisting qualities. There is little or no tendency for the limestone concrete to spall or crack and its insulating value is generally greater. In fires of long duration the limestone aggregate near the surface becomes calcined and in some cases necessitates more surface repair to the protective covering than where trap rock is used, but these cases are the exception rather than the rule. So far as tests have been made it has been found that for rocks of a given mineral composition those of coarsely crystalline structure are not as resistant to fire as those of fine structure. Broken bricks or burnt clay aggregates give favorable results in strength and fire resistive properties.

Thick sections are more resistant to fire than thin ones. However, when the same thickness of protective materials are used the thinner sec-

tions will have a larger gross area in proportion to the effective structural section than the thicker ones.

Protective coverings should be proportioned in accordance with the fire resistive qualities of the materials and the exposure conditions of the member protected. Free standing columns and deep girders require equal thicknesses of protection, beams of secondary importance or of small size may require somewhat less and slabs least of any of the ordinary members. The thickness of protective covering for columns, girders and other equally important members should not be less than 1½ in. unless the fire resistance characteristics, the materials and construction, and their conditions of service are known to warrant less.

DETAILS OF DESIGN.

Certain admonitions regarding the design of reinforced-concrete buildings may be in order here.

Columns.—As far as possible use round columns if the aggregates used show tendency to spall in fires. If square or oblong columns are used the corners should be chamfered. The vertical reinforcement of columns should be thoroughly hooped or tied, and should be rigidly held in place in the forms so as to be imbedded to a depth of 1½ in. from the surface. When conditions require it, as, for instance, where extra fire hazards exist, an extra inch of material all around columns is needed.

Girders and Beams.—The bottoms of girders and important beams should have 1½ in. of protective concrete below the reinforcement. Additional covering will be required where extra fire hazards exist the same as for concrete columns. The embedment of the reinforcement from the side of these members should be not less than 1½ in. Closely-spaced stirrups in the region of the beam where the reinforcement is subjected to compressive stresses will be helpful in cases of fire.

Less protective material may sometimes be allowable in unimportant beams, not because they are more fire resistant but because the consequences of damage by fire may not be so serious.

Slabs.—The protective covering to the reinforcement of slabs and walls should be ¾ in. when non-spalling aggregates are used. The use of aggregates which spall in fires makes it advisable to have not less than 1 inch of protective covering. Where the probability of prolonged fires is present use a greater thickness of protective concrete and avoid covering having spalling types of aggregates altogether unless it is effectively bonded to the slab with a secondary reinforcement of metal fabric. The greater thickness of concrete of the kind that spalls is not necessarily more effective unless it is so bonded that spalled material can not fall away. Avoid tying thick and thin sections of concrete together in such a way that rapid heating will cause rupture from unequal expansion.

Expansion Joints.—The spacing of expansion joints close enough together to prevent excessive stresses in columns and floors from unequal heating will result in less damage in cases of severe fires.

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85 Reinforced Concrete for Construction of Fire-Resisting Buildings, by J. Sheppard.
Concrete and Const. Eng. (London), Nov., 1907.

86 Investigations of Thermal Conductivity of Concrete and Embedded Steel, by I. H. Woolson.
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Eng. N., v. 58, p. 166, Aug. 15, 1907, June 28, 1906.
Gives results of fire tests on reinforced concrete of cinder, gravel and trap aggregates.

87 Safety Against Fire Elongation Due to Temperature and Heat-Conductivity of Reinforced Concrete, by E. Suenson.
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88 San Francisco Fire.
Report by Committee of Twenty of the Natl. Bd. of Fire Und.
Report by S. A. Reed, Cons. Engr., to Natl. Bd. of Fire Und.

89 San Francisco Earthquake and Fire, A. L. A. Himmelwright, the Roebling Const. Co.

90 Trial by Fire at San Francisco, Natl. Fireproofing Co.

91 San Francisco Earthquake and Fire of April 18, 1906, and their effects on structures and structural materials, by Gilbert, Humphrey, Sewell and Soule.
Bulletin 324, U. S. Geol. Survey, 1907.
Trans. A. S. C. E., v. 58, p. 206, 1907.
Effect of fire on building materials noted. Reptd. by Genl. Committee of Six Special Committees of S. F. Assoc. of A. S. C. E., with discussion.

92 Fire and Load Test of Beams, by H. B. MacFarland.
Eng. Rec., v. 55, p. 380, March, 1907.

93 Standard Test for Fireproof Floor Construction.
Proc. A. S. T. M., v. 7, p. 179, 1907; v. 8, p. 210, 1908.

94 Fire Resistance of Reinforced Concrete Construction, by S. E. Thompson.
Concrete Eng., June, 1907, p. 261.

1908 95 Recent Experiences with Reinforced Concrete and Concrete Block Buildings in Fires.
Engr.-Contr., July 8, 1908.

96 Concrete is Fireproof.
Concrete Engineering, July, 1908.

97 Lessons from Fire in Reinforced Concrete Factory Building.
Iron Age, May 28, 1908.

98 Concrete in Case of Fire.
Manufacturers' Record, June 4, 1908.

99 Notes on Fireproof Construction—Advantages of Construction, by C. A. P. Turner.
Cement Era, May, 1908.

100 Reinforced Concrete and Fire Resistance.
The Builder (London), May 16, 23, 1908.

101 Fireproof Building Construction and Concrete Building Construction.
Eng. N., June 11, 1908.
Abs. of 2 papers before N. F. P. Assn., 2800 w.

102 Fire of the Dayton Motor Car Works, by J. B. Gilbert.
Eng. Rec., v. 57, p. 384, March 28, 1908.
Effect on concrete buildings noted. 2000 w. (Reinforced bldg.).

103 Cinder and Stone Concrete Under Fire, by G. B. Waite.
Trans. A. S. C. E., v. 60, p. 470, 1908.

104 Fireproof Qualities of Concrete Partitions, by I. H. Woolson.
Cement Age, v. 6, p. 578, June, 1908.

105 Concrete in the Chelsea Fire, by S. E. Thompson.
Cement Age, v. 6, p. 569, June, 1908.

106 Recent Fire Tests of Building Materials.
Cement Age, v. 7, p. 401, Dec., 1908.

107 Concrete Fireproofing.
Cement World, August, 1908.

108 Fireproof Properties of Concrete.
Cement Age, v. 6, p. 555, 1908.

109 Tests on Effect of Heat on Reinforced Concrete Columns.
Eng. N., Sept. 20, 1908.
Work of the Chicago Laby. of the Natl. Fireproofing Co.

110 Tests on Concrete for Heat Resistance, by F. W. Scheidenhelm.
Street Ry. Journal, 1908.
Abs. Eng. Digest, v. 5, p. 297, March, 1908.
Chem. Abs., v. 2, p. 1610, 1908.
Tests to determine adaptability of concrete for smoke flues. Blocks made of (1) cement and sand, (2) cement, sand and lime, (3) cement, sand and cinders. Exposed to boiler flue heat 84 hours, temp. at end of test being 800 deg. F. Block (1) showed no deterioration, (2) softened, while (3) showed signs of burning.

111 Report of Fire, Load and Water Test on Triangular Reinforced Concrete Floor System.
Test No. 60, Printed Report No. 9, Fire Testing Station, Columbia University, 1908.
Tests conducted by I. H. Woolson, in co-operation with city building bureau of New York. Built by Amer. Steel and Wire Co.

112 Concrete and Fire Resistance. Some Comparative Tests.
Concrete and Const. Eng., Nov., 1908.

113 Status of Reinforced Concrete from Fireproof Standpoint, by E. G. Perrot.
Cement Age, v. 7, p. 415, Dec., 1908.

114 Report of Fire in Parker Building, by W. C. Robinson.
New York Board of Underwriters, April 22, 1908.
Eng. N., v. 59, p. 567, May, 1908.
Results of investigations, recommendations, photographs of damage to this building. Columns were of cast iron, with porous tile fireproofing, and slabs were of tile arches on I-beams. Building failed completely.

1909 115 Fire Resistive Properties of Various Building Materials, by R. L. Humphrey.
Bulletin 370, U. S. Geol. Survey, 1909.
Cement World, v. 3, p. 257.
Chem. Abs., v. 3, p. 2212.

116 Fire-Resistant Construction of Buildings, by Wilson, Lesley, Humphrey and Whinery.
Trans. A. S. C. E., v. 65, p. 274, Dec., 1909.
Discussion of fire-proof constructions, loss by fire, insurance, and methods of reducing fire loss.
Proc. A. S. C. E., Sept., 1909, discussion, 6500 w.

117 Practical Fire Test of a Reinforced Concrete Factory.
Cem. Age, Aug., 1909, 800 w.

118 Heat Conductivity of Concrete.
Proc. Nat'l Fire Prot. Assn., 1909, p. 116.

119 Resistance of Concrete to Fire.
Revue Generale, Oct. 20, 1909.

1910 120 Fire Resistance of Reinforced Concrete Grain Tanks.
Concrete Eng., Feb., 1910.

121 Concrete as Protection Against Fire.
Insurance Eng., Jan., 1910.
Abs. Cement Age, v. 10, p. 208, March, 1910.

122 Fire Protection in San Francisco Under New Building Law—Features that Relate to Cement Construction.
Eng. News, Feb. 10, 1910.

123 Concrete Withstands a Hot Fire.
Rock Products, May 22, 1910.

124 Fire Test on a Mushroom System Building.
Concrete Eng., Oct., 1910.

125 Heat Conductivity of Concrete.
Eng. N., 1910, I-575.

126 Remarkable Instance of Heat Transmission through Concrete.
Eng. N., 1910, II-470.

127 Reinforced Concrete Fireproof Walls, by R. Saliger.
Beton u. Eisen, No. 14, 1910.
Reference to American tests, and those at Vienna.

128 Fires and Fire Tests.
A selection of papers and reports, 3 vols., of the British Fire Prevention Committee, 1910. (A) Fires and Prevention; (B) Floors and Partitions; (C) Doors, Shutters and Glazing.

1911 129 Fire Effects on Building Materials.
Jour. A. S. M. E., May, 1911, 15000 w.
Discussion of Gilbreth's paper.

130 Der Brand des Warenhauses Enders in Dresden am 13 August, 1911.
Bet. u. E., 1911, S. 353.

131 Veroff des Deutscher Ausschusses fur Eisenbeton, Heft. 11 and 26.
Results of Investigation of Fire Damage in 1910.

132 Thermal Properties of Concrete, by C. L. Norton.
Proc. Nat. Assn. Cem. Users, v. 7, p. 78, 1911.
Study of physical properties of portland cement concrete which affect its value as fire resistant material.

1912 133 Fire Resistance of Concrete and Reinforced Concrete, E. O. Sachs.
Proc. Int. Assn. Test. Mat., Paper XXVII-1, Part II, 1912.
Gives definitions and requirements for fireproof members.

134 Crushed Limestone in Concrete as a Fireproofing Material, by R. J. Wig.
Bull. 14, Natl. Lime Mfg. Assn., Feb., 1912.
Chem. Abs., v. 7, p. 1273, 1913.
Reviews concrete tests by fire, and concludes that experimental data does not show superiority of gravel over limestone as an aggregate.

135 Fire Resisting Concrete, by H. G. Holt.
Concrete and Const. Eng., v. 7, p. 13, 1912.
Compares fire behavior of concrete and tile and shows causes of failure of latter. Chief advantage of concrete is its equal expansion with steel. Thermal conductivity depends largely upon aggregate used.

136 Fire Tests of Floors in the United States, by Woolson and Miller.
Proc. Int. Assn. Test. Mat., Paper XXVII-2, 2nd Sect., 1912.
Tests made on cinder, slag and stone concrete, hollow tile, etc., both plain and reinforced.
45 Tests of Cinder Concrete.
23 Tests of Stone Concrete.
2 Tests of Slag Concrete.
2 Tests of Gravel, (protected soffit).
3 Tests of Mortar.

1913 137 Fire Tests of Partitions for Buildings, by H. B. McMaster.
Architectural Eng., v. 32, p. 84, 1913.
Describes tests with expanded metal lath with cement plasters.

138 Experience with Concrete in Fires—Reviews from Baltimore and San Francisco Fires. G. E. Fisher.
Jour. A. S. M. E., June, 1913, 2500 w.

139 Fireproof Building Construction, (Reinforced Concrete, etc.), G. McCook.
Applied Science, May, 1913.

140 Tests of Fireproof Construction for City of New York.
School of Mines Quarterly, April, 1913. 2500 w.
Methods on full size construction and samples.

141 Fireproof Construction—Testing Materials for their Power of Resisting Extreme Temperatures. H. Perrine.
Sci. Am. Supp. 75: 360-2, June 7, 1913.

142 Review of Concrete as a Fire Resistant.
Concrete-Cement Age, v. 3, p. 161, Oct., 1913.

143 Thermal Properties of Concrete, by C. L. Norton.
Jl. A. S. M. E., v. 35, p. 1011, June, 1913.
Ind. Eng., v. 13, p. 368, Aug., 1913.

144 Fire, Load and Water Test upon Cinder Concrete Floor Arches, by H. Perrine and A. Oliver.
Sci. Amer., v. 109, p. 115, Aug. 9, 1913.
Includes reinforced concrete floor arches tested at Columbia Fire Station.
Eng. N., Aug. 21, 1913.
Cement, Sept., 1913.

1914 145 Determination of Mineral and Rock Densities at High Temperatures, by Day, Sosman and Hostetter.
Am. Jl. Science, v. 37, January, 1914.

146 Heat Conductivity of Fireproofing Building Materials, by Heyn, Bauer and Wetzel.
Mitt. Kgl. Materialpruf., v. 32, p. 89, 1914.
Stahl u. E., v. 34, p. 832, 1914.
Chem. Abs., v. 9, p. 519.

147 Fire Resistant Construction and Equipment of Buildings.
Eng. N., 71: 1091-2, May 14, 1914.

148 Fire Resisting Qualities of a Concrete Building at Salem, Mass.
Concrete-Cement Age, v. 5, p. 61, August, 1914.

149 Real Fireproof Building.
Eng. N., 72: 154-5, July 16, 1914.

150 Reinforced Concrete Survivor of the Salem, Mass., Fire.
Eng. N., II-154, 1914.

151 Fireproof Buildings, by E. R. Hardy.
Arch. and Bldg., v. 46, p. 327, August, 1914.

152 Fireproofing of Buildings, by Kinney, Wason and Godfrey.
Concrete-Cement Age, v. 5, p. 155, Oct., 1914.

153 Service Fire Test of a Fire Proof Building, Amoskeag Savings Bank, Manchester, N. H.
Eng. N., 71: 537, March 5, 1914.

154 Behavior of Reinforced Concrete in Fires.
Eng. Rec., v. 70, p. 635-6, Dec. 12, 1914.
Describes fires in three reinforced concrete buildings and their ability to withstand fire.

155 Disastrous Fire at Edison Factory.
Eng. Rec., v. 70, p. 660, Dec. 19, 1914.
Photographs and descriptions of damage to reinforced concrete buildings as compared with other types.

156 Rapid Destruction of Fireproof Buildings (Edison Fire), by S. H. Bunnell.
Iron Age, 94: 1381-3, Dec. 17, 1914.

1915 157 Fire Resisting Qualities of Concrete, by W. M. Kinney.
Concrete-Cement Age, v. 6, p. 40, January, 1915.

158 Edison Fire—Miscellaneous reports as follows:
Eng. and Contr., 43: 193-6, Mar. 3, 1915.
Eng. and Contr., 43: 145, Feb. 17, 1915.
Concrete-Cement Age, 6: 120-1, March, 1915.
Eng. Rec., 71: 239-42, Feb. 20, 1915.—Underwriters' and Fire Prot. Assn. Report.

159 Concrete Construction and Edison Fire, L. C. Wason.
Textile World, v. 48, p. 537, February, 1915.

160 Concrete Melted in Edison Fire.
Eng. N., v. 73, p. 362, Feb. 18, 1915.

161 Hot Chemicals and Concrete Form Slag at Edison Fire.
Eng. Rec., 71: 184, Feb. 6, 1915.

162 Fire Protection of Structural Members, by S. Broadbent.
Concrete-Cement Age, v. 6, p. 124, March, 1915.

163 Report of Committee on Edison Fire.
Jl. Amer. Concrete Inst., v. 3, No. 8, August, 1915.
Photographs. Conclusions of committee as to fireproof structures and lack of conformity with standard practices.
Abs. Engr. and Contr., 43: 193-6, Mar. 3, 1915.
Abs. Engr. and Contr., 43: 145, Feb. 17, 1915.
Concrete-Cement Age, 6: 120-1, March, 1915.

164 Proposed Series of Fire Resistance Tests of Columns.
Eng. Rec., v. 72, p. 800, Dec. 25, 1915.
Iron Trade Review, v. 58, p. 823, April 13, 1916.
Describes tests to be made at the Underwriters' Laboratories on centrally loaded columns of reinforced concrete, etc.

1916 165 How Concrete Stands Fire.
Factory, v. 17, p. 480, October, 1916.

166 Brandenproben an Eisenbetonbauten, 1914-1915.
Deutscher Ausschuss fur Eisenbeton, Heft 33, 66 pp., 1916.
Elaborate series on test buildings, involving such points as the following: Aggregates were 3 gravels, pumice, granite basalt, blast furnace slag, sandstone, etc. Structures involved test of walls, stairs, loaded beams, etc. Study covered behavior and resisting power of buildings during fire, effect of repeated fire, change in length of beams, transmission of heat through concrete, effect of thickness of covering, comparison of basalt and granite aggregates, influence of heat on tensile strength and elongation, influence of heat on properties of twisted square steel, strength of concrete before and after fire, behavior of buildings during tearing down. 54 illustrations.

167 Fire or Explosion Wrecks Part of Concrete Warehouse at Far Rockaway, N. Y.
Eng. News, v. 76, p. 956-8, Nov. 16, 1916.
Tells of damage to concrete beams and columns by fire.

1917 168 The Fire at the Venesta Works, Silvertown, London, on Jan. 19, 1917.
Red Book 211 of the British Fire Prev. Comm.

169 Fire at Millennium Mills, Victoria Docks, Silvertown, London, on January 19, 1917.
Red Book 208 of the British Fire Prev. Comm.

170 Concrete Structure Wrecked by Intense Heat at Quaker Oats Co. Fire, Causing $2,000,000 damage.
Eng. N., 78: 17-21, April 5, 1917.

171 The Fire at the Quaker Oats Co. Premises, Peterboro, Ontario, on December 11, 1916.
Red Book 225, British Fire Prev. Comm., 1918.
Two reports, by A. J. and T. D. Mylrea.

172 Effect of Fire on Reinforced Concrete Building Analyzed.
Eng. N., v. 78, p. 633, June 28, 1917.

173 Concrete Grain Tanks Resist Fire.
Concrete, v. 11, p. 34, Aug., 1917.

174 Fire Resistance of Concrete, by W. A. Hull.
Concrete, v. 11, p. 56, August, 1917.

175 New Grading of Fire Resistance as Measured by Tests.
Eng. N., v. 78, p. 435, May 31, 1917.
Work of Joint Committee.

176 Fire-Wrecked Concrete Building Repaired for Service.
Eng. N., 79: 222-3, Aug. 2, 1917.

177 Reports of Tests Made by British Fire Prevention Committee from 1897 to date.
List No. 210—Publications of Brit. Fire Prev. Comm., 1917. Red Books include tests on concrete floors, columns, walls, etc., using many aggregates, types of construction and other variables.

178 Quartz-Gravel Aggregate Main Cause of Fire Damage to Concrete Building at Far Rockaway, N. Y. I. H. Woolson.
Eng. N., v. 78, p. 82, Jan. 11, 1917.
Eng. Rec., v. 75, p. 98, Jan. 20, 1917. Discussion p. 356, March 3, 1917, and p. 399, March 10, 1917.
Concrete, v. 10, p. 48, February, 1917.

179 Quartz-Gravel Concrete and Fires, by N. C. Johnson.
Eng. Rec., v. 75, p. 399, March 10, 1917.

180 Effect of Fire on Flat Slab Building of Quaker Oats Co., Peterboro, Ont., Dec. 11, 1916, by T. D. Mylrea.
Jl. West. Soc. Eng., v. 22, p. 509-39, Oct., 1917, Discussion 539-45. 26 plates.

181 Fire and Load Tests of Columns Now Being Made.
Eng. N, v. 79, p. 215, Aug. 2, 1917.
Describes tests at Underwriters' Laboratories on concrete, columns, etc.

182 Aggregate is Critical Element in Fire-Resistance of Concrete Columns, by W. A. Hull.
Eng. News, v. 81, p. 49, July 4, 1918.
Concrete, v. 13, p. 51, Aug., 1918.

1918 183 Fire Tests of Concrete Columns, by W. A. Hull.
Proc. Am. Concrete Inst., v. 14, 1918.
Tests made on columns of different shapes and reinforcement, and using different aggregates.

1919 184 Second Report on Work of Underwriters' Laboratories.
American Architect, v. 115, p. 829, June 11, 1919.

185 Behavior of Reinforced Concrete Columns Under Fire Test, by W. A. Hull.
Amer. Archt., v. 116, p. 561 and 589, Oct. 29, Nov. 5, 1919.
Concrete, v. 16, p. 172, April, 1920.

186 Comparison of Heat Insulating Properties of Some Materials Used in Fire-Resistive Construction, by W. A. Hull.
Tech. Paper 130, U. S. Bur. of Standards, Nov. 12, 1919.
Abs. Jl. Franklin Inst., v. 188, p. 558, Oct., 1922.
Tests made on concrete of 1-2-4 and 1-3-6 mixes, using trap, clinker, cinders, limestone and gravel.

187 Fireproofing Concrete Columns with Plates, by W. A. Hull.
Eng. News-Rec., v. 83, p. 92, July 10, 1919.

188 Fire Tests of Concrete Columns, by W. A. Hull.
Proc. Amer. Concrete Inst., v. 15, p. 89, 1919.
Tests made using different gravels, slag and trap, in round and square columns. Cement plaster also substituted for protective concrete.

189 Report of Committee on Fireproofing, by W. A. Hull.
Proc. Amer. Concrete Inst., v. 15, p. 314, 1919.
Engr. and Contr., v. 52, p. 105, July 23, 1919.
Eng. News-Rec., v. 83, p. 76, July 10, 1919.

190 Crosby-Fiske-Forster Handbook of Fire Protection, 1919, 757 pp.

1920 191 Fire Tests of Building Columns.
Underwriters' Laboratories, Chicago, December, 1920.
Fire and Strength tests of brick piers.

192 More Light on Protecting Concrete Against Fire.
Eng. News-Rec., v. 84, p. 469, March 4, 1920.

193 National Fire Protection Assn. Gives New Recognition to Concrete.
Concrete, v. 16, p. 278, June, 1920.

194 Fire Tests on Reinforced Concrete Construction, 1914-15, by M. Gary.
Deutscher Ausschuss fur Eisenbeton, v. 33, p. 66, Berlin, 1916.
Concrete and Const. Eng., v. 15, p. 345, and p. 406, 1920.
Results of tests on two old concrete houses and two new houses built especially for this purpose. Photographs and results.

195 Causes of Failure of Concrete in Fire, by A. M. Schoen and I. H. Woolson.
Concrete, v. 17, p. 35, July, 1920.

196 Fire Tests of Concrete Columns, by W. A. Hull.
Proc. Am. Concrete Inst., v. 16, p. 20, 1920.
Abs. Amer. Archt., 1920.
Abs. Concrete and Const. Eng., v. 15, p. 187 and 262, 1920.
Fire tests made on columns of various designs and reinforcing.

197 Report of Committee on Fireproofing.
Proc. Am. Concrete Inst., v. 16, p. 186, 1920.
Canadian Engr., v. 38, p. 505-06, May 27, 1920.
Engr. and Contr., v. 53, p. 717, June 23, 1920.
Recommendations for changes in previous report.

198 Effect of Fire on a Concrete Warehouse at Galveston.
Eng. N., 85: 980-3, 1101-2, 1298, 86: 353 N18, Dec. 2, 30, 1920. Feb. 24, 1921.

1921 199 Fire Resistance of Building Columns, by Ingberg, Griffin, Robinson, and Wilson.
Report of Underwriters' Laboratories, Chicago, 1921.
Tech. Paper 184, U. S. Bur. of Standards, 1921.
Associated Factory Mutual Fire Insurance Companies, National Board of Underwriters and Bureau of Standards, (separate publication).

Abs. Engr. and Contr., v. 55, p. 510, May 25, 1921.
Eng. News-Rec., v. 87, p. 106, 145, 292, 1921.
Concrete, v. 18, p. 267, June, 1921.
Eng. and Contr., v. 59, p. 941, April, 1923.
Rock Products, v. 26, p. 120, March 10, 1923.
Elaborate series of tests on full size building columns, carried out jointly by Assoc. Factory Mut. Ins. Cos., Natl. Bd. of Underwriters, and the Bur. of Standards.

200 Fire Prevention and Fire Protection as Applied to Building Construction. Jos. K. Freitag.
Book published in 1912, revised 1921.

201 La Resistance a l'Incendie des Differentes Genres de Piliers et Colonnes, by P. Calfas.
Genie Civil, v. 79, p. 261, 283, Sept. 24 and Oct. 1, 1921.
Foregoing is abstract of column tests at Underwriters' Laboratory.

202 Effect of High Temperature Fire on Concrete Buildings.
Eng. News-Rec., v. 86, p. 760, May 5, 1921.

203 Effect of Fire on Concrete.
Sci. Am., v. 125, p. 223, Sept. 24, 1921.

204 Behavior of Concrete Constituents Under Fire, by S. H. Ingberg.
Proc. Amer. Concrete Inst., v. 17, p. 234, 1921.
Discusses fire resistance of cement, fine and coarse aggregates in concrete.

205 Report of Committee on Far Rockaway Fire.
Proc. Amer. Concrete Inst., v. 17, p. 368, 1921.
Photographs, description of damage and restoration.

206 Lessons in Fire Resistance from Frankford Fire, by W A. Hull.
Proc. Am. Concrete Inst., v. 17, p. 205-12, 1921. Ill.
Requirements learned from this conflagration.

207 Fire Resistance of Concrete and Reinforced Concrete, D. W. Wood.
Jl. Junior Inst. Eng., v. 31, part 10, July, 1921.
Concrete and Const. Engr., v. 16, p. 660, Oct.; p. 722, Nov.; p. 799, Dec., 1921, and v. 17, p. 29, Jan., 1922.
Strength and fire tests on concrete slabs. Used gravel, sandstone, limestone, granite, slags, coke breeze, fire brick.

208 Report of Tests of British Fire Prevention Committee.
List No 10, 1921. Gives results of various tests of slabs.
Some of the Red Books (the reports of this committee), are briefly listed in following groups. (C denotes concrete, RC denotes reinforced concrete).

Fire Tests.

C 101	RC 229	C 243	C 216
RC 221	RC 231	RC 244	C 217
RC 222	RC 232	RC 246	C 218
RC 223	RC 234	RC 248	C 219
C 224	C 237	C 249	
RC 226	RC 238	RC 250	258—Synopsis
RC 227	RC 239	C 212	
C 228	RC 242	C 213	

Heat Conductivity 251 and 252.

Mineralogic and geological data on aggregates 256.

Mechanical Tests of Concretes 257.

1922 209 Report of Fire Tests Made on Building Columns.
Laboratories Data (Underwriters'), v. 3, p. 182, 1922.
Fire tests made at Underwriters' Laboratories.

210 Germans Learn Lessons from First Concrete Building Fire.
Eng. N., 88: 795-6, May 11, 1922.

211 Resistance to Fire of Concrete and Reinforced Concrete, by Lea and Stradling.
Engineering, v. 114, p. 341 and 380, Sept. 15 and 22, 1922; also p. 337 and 363.
Consider possibility of making concrete which will retain strength during and after exposure to high temperatures, and possibility of preventing steel from reaching temperature at which strength is reduced to or below that required to carry loads. Describes tests of specimens in furnace.

212 Reinforced Concrete and Fire Resistance, J. Singleton-Green.
Concrete and Const. Eng., v. 17, p. 579, 645, 723, 789, Sept., Oct., Nov., and Dec., 1922, and v. 18, p. 227, April, 1923.
Gives examples of failures in fires.

213 When is Siliceous Aggregate Objectionable?
Concrete, v. 23, p. 1, July, 1923.

1923 214 Index of May 1, 1923, covering publications of the National Fire Protection Association on the subjects of Fire Prevention and Fire Protection. 76 pp. In addition to a number of references from this book, listed below, the index includes the title and numbers of reports covering fires involving a loss of $500,000 or over for each year from 1910 to 1922 inclusive. There are also references to particular reports of certain fires at the places and times as follows: Aalborg, 1922; Alameda, Calif., 1920; Averne, L. I., 1922; Astoria, Ore., 1922; Athens, Ga., 1921; Atlanta, Ga., 1908, 1921; Augusta, Ga., 1916, 1921; Baltimore, Md., 1904; Bangor, Me., 1911; Boston, 1872; Chelsea, Mass., 1908; Chicago, 1922; East Boston Docks, 1908; Evansville, Ind., 1910; Grandview, Tex., 1920; Jamestown, Kansas, 1911; Manila, 1921; Nashville, Tenn., 1916; New Orleans, La., 1908; Paris, Tex., 1916; Pittsburgh, Pa., 1917; Salem, Mass., 1914; Superior, Wis., 1907; West Lynn, 1906; Worcester, Mass., 1906.

215 Concrete as a Material for Fireproof Construction.
M. C. Tuttle, Nat. Fire Prot. Assn., Q. v. 2, No. 4, p. 415.

216 Napthalin Fire Wrecks Concrete Building.
Nat. Fire Prot. Assn., Q. v. 15, No. 1, p. 80.

217 Concrete Column Failure in Sisal Fire.
Nat. Fire Prot. Assn., Q. v. 14, No. 2, p. 219.

218 Concrete Construction in Fireproof Buildings.
Nat. Fire Prot. Assn., Q. v. 7, No. 4, p. 403.

219 Concrete Grain Tanks in Severe Fire.
Nat. Fire Prot. Assn., Q. v. 11, No. 2, p. 137.

220 Recent British Fire Tests (Concrete).
Nat. Fire Prot. Assn., Q. v. 15, No. 3, p. 265.

221 Concrete Weakened by Fire (Editorial).
Nat. Fire Prot. Assn., Q. v. 4, No. 2, p. 153.

1922 222 Fire Resistance of Concrete and Reinforced Concrete, D. W. Wood.
Concrete and Constructional Engineering, v. 16, Nos. 10, 11 and 12, Oct., Nov., and Dec., 1922.

223 The Behavior of Concrete and Reinforced Concrete when Exposed to Fire. (Beton und Eisenbeton in Feuer.)
M. Gary. Beton u. Eisen, v. 21, No. 3, Feb. 10, 1922.

1923 224 Fire Resistance of Concrete Columns, S. H. Ingberg.
Engineering and Contracting (Buildings), v. 59, No. 4, April 25, 1923.

1924 225 Cinder Concrete Units, Fire and Water Tests, A. H. Beyer.
Concrete, 24: 25-7, Jan., 1924.

226 How Structures Withstood the Japanese Earthquake and Fire, H. M. Hadley.
Amer. Concrete Inst. Proc., 20: 188-209, 1924.

227 Fire Resistance of Concrete Building Units.
Tests made at the Underwriters' Laboratories in Chicago. Book of 100 pp., illus. Reprinted by the Portland Cement Assoc. Details of these tests are set forth in the Committee Reports in vols. 19 and 20, Proc. of the Amer. Concrete Inst.

228 Protective Concrete Covering.
Report of Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, Chapter VIII-B, Secs. 67-68, Proc. A. S. C. E., Oct., 1924, p. 1180.

1924 229 Concrete Storehouse, Naumkeag Steam Cotton Co., Salem, Mass. (Conflagration).
7 pp. Booklet by the Inspection Department of the Associated Factory Mutual Fire Insurance Companies. Building was erected in 1906; 100 x 56 ft., 4 stories. Reinforced concrete trap rock aggregate. Brief description and pictures. Fire was on exterior and did not enter building. No damage to contents. Only a slight spalling of concrete in few spots on one side of building, and only about 2 inches of reinforcing rod exposed.

DISCUSSION.

Mr. Lindstrom.

Robert Seth Lindstrom.—I am glad that this committee brought out the feature of limestone as an aggregate. Several years ago I read a paper before the convention of Building Commissioners of the United States, held at the Auditorium Annex, Chicago. After the paper was read, I referred to different aggregates in a fire-resistive manner. The chairman of the committee asked me if I would sit in on the committee meeting, to which I consented, and was asked different questions regarding building materials, etc. Being an architect by profession, I knew something about it, and at that committee meeting it developed that there were 36 building commissioners present from different parts of the United States. The commissioner from New York City was very much interested and wanted to find out why in New York City all the old specifications of architects advocated traprock aggregate for reinforced-concrete floors. The question resolved itself down to the point that there were several million dollars worth of building permits held up by this commissioner until he had attended this convention to find out whether he could change his attitude regarding aggregates and not use traprock exclusively. At that time the Underwriters' Laboratory, at Chicago, was carrying on a series of tests of several types of columns, and it just so happened that this committee meeting was in the morning, and in the afternoon of that day the fire test was to be carried at the laboratory, with a 50-ton load on top of a certain sized traprock concrete column for comparison with other types of columns using other than traprock concrete. I telephoned the laboratory and we all went there in a body, and to the surprise of myself and the commissioners present, this traprock concrete column by a fire test similar to others, in spalling popped like pop-corn. That changed the attitude of the commissioner of the city of New York, and it appeared that in a four-day previous similar test we were told that the limestone aggregate concrete column had been tested to the full eight-hour period and continued from the eight-hour period to the 24-hour period, and I think spalled a little at the top at the 36-hour period of test. It was a longer period than any other column tested and it stood up. It was explained to us by the laboratory attendants that due to the projection of the reinforcement at the top of the column meeting the red-hot plate on top, it bent the rods a little bit and caused spalling at top of column. They sawed this limestone aggregate column in two, and the column inside was found perfect. The building commissioners went back with the assurance that regardless of theory, actual practice had proven to them that other aggregates than traprock were better for reinforced-concrete construction.

Mr. Woolson.

Ira H. Woolson.—I am afraid the last speaker may have inadvertently left a wrong impression before the association as to the behavior of traprock under fire attack. He said that in this particular example which his committee witnessed, the traprock popped like pop-corn, or something

of that kind. Some years ago it was my privilege to conduct fire tests upon reinforced concrete constructions in a rather elaborate way for the city of New York, over a period of ten years. I suppose during that time I conducted four-hour fire tests (that was the standard test of the city of New York), upon probably twenty, maybe more, reinforced-concrete floors and some columns of small size which had been constructed of traprock, and I never saw traprock perform that way. It does crack and spall as all concrete will do to a certain extent, but as to a popping action, or any excessive disintegration during the fire, I never saw it, and I have a great many photographs showing the results of those tests. In my opinion, of the aggregates with which we are familiar, traprock stands next to limestone as the best aggregate to withstand fire. It is used in the city of New York and vicinity more than elsewhere because it is an available aggregate in that region. The behavior of concrete itself during the fire, so far as spalling is concerned, is no greater than for limestone. The limestone calcines on the surface, and that calcined material is a good retardant of heat, and consequently keeps the heat away from the reinforcing material.

After the application of water which usually follows these fire tests, the surface of the limestone concrete is badly scored, washing away the calcined material, and usually will require rather major repair. The traprock will usually retain its form and shape almost perfect with the exception of such limited spalling as may occur due to unequal expansion. Heat goes through traprock concrete somewhat more readily than it does through limestone concrete and, consequently, as a whole, the limestone gives a better result so far as transmission of heat to the steel is concerned. I had rather a humiliating experience which I have related before, perhaps not to this audience, and I might as well make a confession here, in reference to these two particular aggregates. I was conducting a fire test upon a floor and it became necessary to put a four-inch partition across the fire chamber in order to accommodate the size of that chamber to the particular spans between beams of the system that was under test. The fire chamber was about 20 ft. wide by 18 or 20 ft. the other way, so the partition was 18 or 20 ft. long, and about 10 ft. high and 4 inches thick. It was simply a reinforced concrete wall put there, not for fire test purposes, but simply as a barrier. I gave instructions to have it made of traprock and while attending to my duties at the university, the work was begun. During the day I went to inspect it and found the partition about half erected and discovered that the aggregate being used was limestone and not traprock.

I was very much alarmed, because at that time I knew nothing of the superior non-conducting properties of limestone concrete. I stopped the work, hurried a truck to get some new aggregate so as not to spoil the bond, and this was secured and the work continued. We ran a four-hour fire test, and after the fire, water was applied which sprayed down over this partition, of course, as it usually does, very vigorously, a heavy

stream of water applied for five or ten minutes. Upon inspection it was discovered that the limestone portion of that partition was washed away to a depth perhaps varying from a half inch to an inch. No measurements had been made of the temperature that had passed through that partition. I knew nothing about that; that point did not occur to me. The traprock portion of the partition was as it was when put in. The fins between the boards and the picture of the grain of the wooden forms and everything was just as perfect and nice as it could be. I have splendid photographs showing the conditions that confirmed my previous opinion that limestone was not suitable for that purpose. Much of the limestone surface was washed away. We knew it would have to be scrapped off, an inch or more, and repaired, whereas the traprock was in beautiful condition.

On the strength of that, a traprock supply company asked me if I would express an opinion as to the behavior of traprock as compared with limestone or aggregates to resist fire. I had no hesitation in doing so, it was a matter of fact as I saw, and I did express an opinion very favorable to the traprock. That is one of those cases where I didn't know fully what I was talking about. The test at the Underwriters' Laboratories a few years later showed that so far as the transmission of heat was concerned, limestone was a much better aggregate. The point I want to bring out is that that traprock partition was not spalled; it did not pop; it was in splendid condition; but I suppose it did transmit more heat than the limestone. I do not want the impression to go that traprock is not a high grade fire resistive aggregate to use. Under conditions where not likely to be subjected to excessively long periods of fire, it might be superior to limestone.

Mr. Clousing.

LOUIS CLOUSING.—We had an experience which probably Mr. Woolson might be able to explain. There was a building that had a settlement, and all the floors were examined. We finally came to the conclusion that there must be something the matter with the foundation. The foundation of this building as well as of the other, was of limestone blocks, and it happened that the boiler had been in that position for a number of years, and the stones, while they were not under the same conditions that they would be in a fire test, had baked so that they were of a chalky consistency and had softened. The temperature was just that of the ordinary boiler set near the foundation of these footings. I do not know what the temperature would be exactly—just the furnace heat of the boiler.

Prof. Woolson.

PROF. WOOLSON.—Was the aggregate limestone?

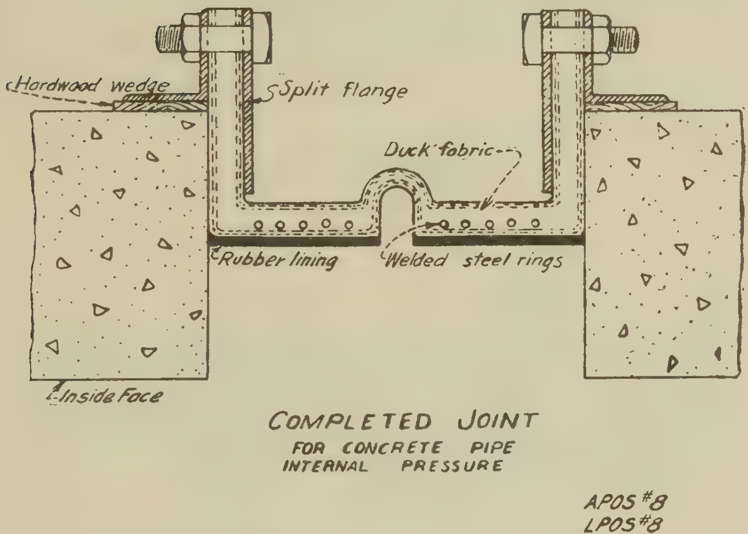
Mr. Clousing.

MR. CLOUSING.—No. Just a block stone of limestone.

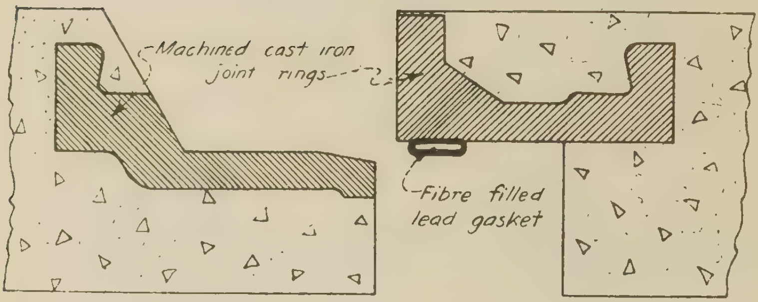
Prof. Woolson.

PROF. WOOLSON.—It was merely a case of long continued calcination, I suppose.

Report of Committee E-8 on Expansion Joints in Concrete Construction.*



*Seven sketches, here reproduced, are presented by R. R. Leffler, secretary, Committee E-8, Expansion Joints in Concrete Construction, to be added to the report of the committee for 1924 (Vol. 20 A. C. I. *Proceedings*) and with that portion of the report beginning at "Force exerted by concrete as temperature changes" (page 619, Vol. 20) constitute the final report of the committee.



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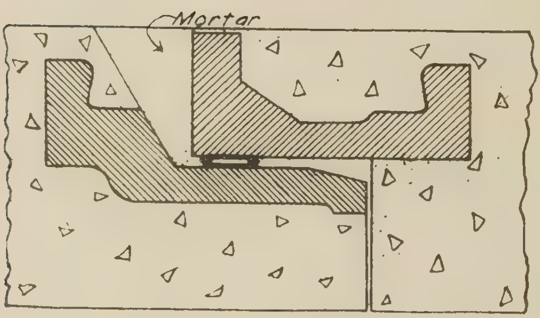
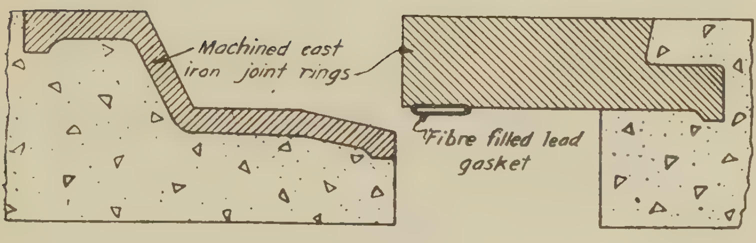


APOS #11
LPOS #11
NPTS #12

COMPLETED JOINT



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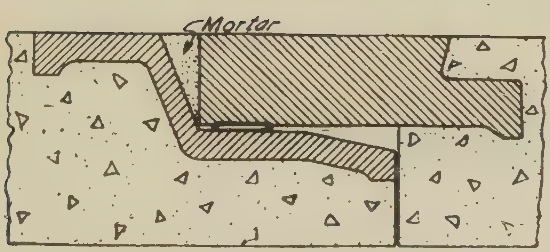
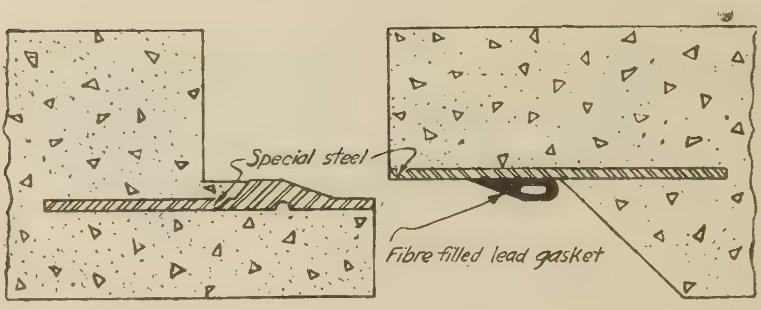


APOS #10
LPOS #10
NPTS #11

COMPLETED JOINT



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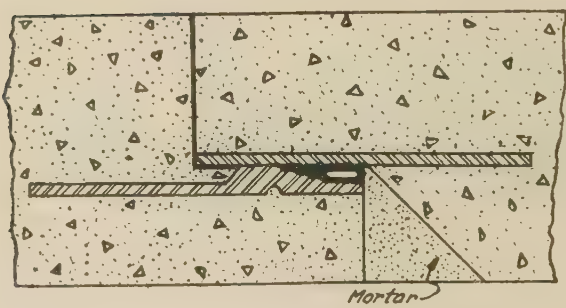
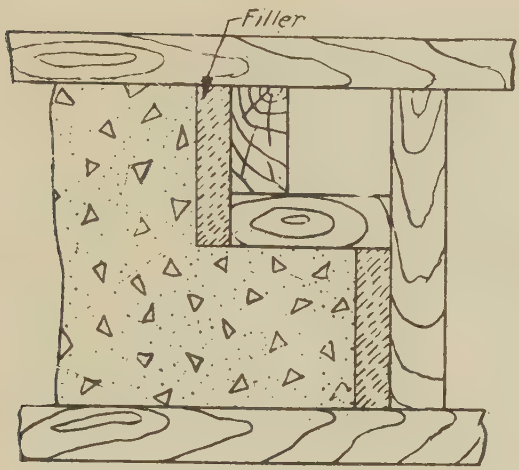


COMPLETED JOINT

APOS #9
LPOS #9
NPTS #10



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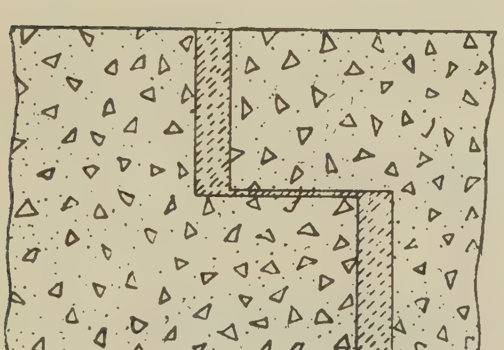
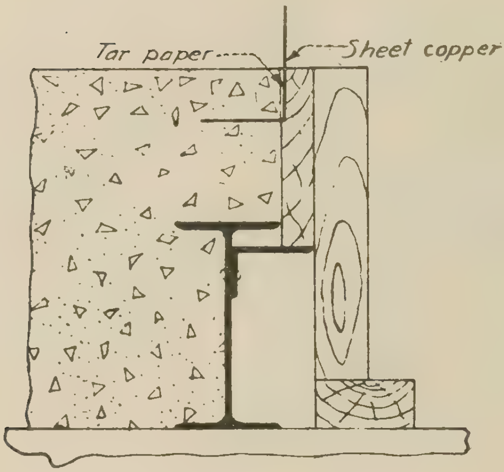


COMPLETED JOINT

GPDS#8
GPTS#3
NPTS#7
NPTSWT#7



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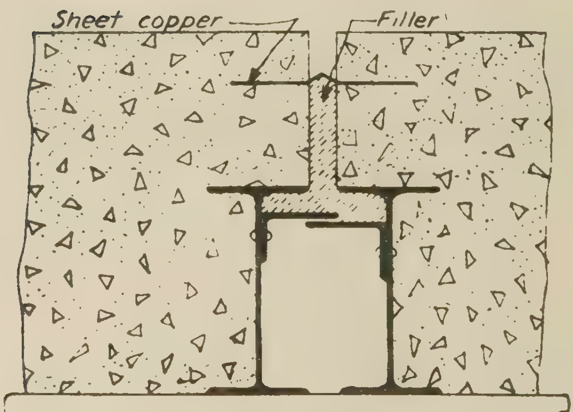
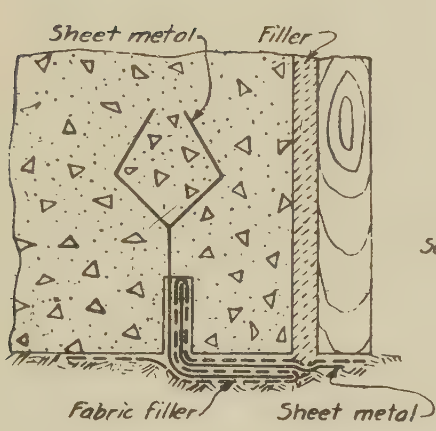


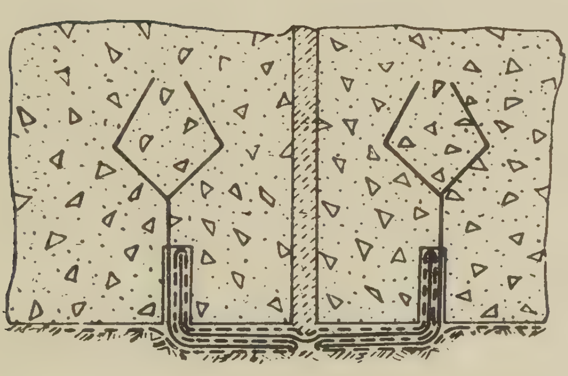
GPOS #9
NPTS #8
NPTSRT #7
NPTSWT #8

COMPLETED JOINT



Suitable for basement floors

INITIAL SET-UP



GPOS#10
NPTS#9
NPTSRT#8
NPTSWT#9

COMPLETED JOINT

Report of Committee J-1: Institute Representation on Joint Committee on Standard Specifications for Concrete and Reinforced Concrete.

The report of the 1924 Joint Committee on Standard Specifications for Concrete and Reinforced Concrete has been mailed to the entire membership of the Institute. Briefly, the report represents a specification based on the work of what is called the new Joint Committee (that is the one organized in 1920) as differentiating it from the joint committee organized in 1904, which presented a final report of the recommendations for use in concrete and reinforced-concrete design and construction, published in 1916. The report of 1916 is specifically stated only to be the basis for a specification. The work of preparing a specification then was set up by joint action of the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering Association, the American Concrete Institute and the Portland Cement Association. That is the formation of the present committee. Since the first meeting in 1920 there have been about 100 sessions from five to six hours each, besides many times that number of the subcommittees.

The Joint Committee has other phases of the art upon which it doubtless will report as the state of the art develops. It can be accepted here only as a report. The report has been out for nearly three years and there has been some discussion on it. It will not go to letter ballot, and its value lies merely in its merits. The Joint Committee would welcome discussion and criticism of its report.

S. C. Hollister, *Chairman.*

The report of the Joint Committee is given on the following pages.

REPORT

OF THE

JOINT COMMITTEE ON STANDARD SPECIFICATIONS

FOR

CONCRETE AND REINFORCED CONCRETE

SUBMITTING

STANDARD SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

Affiliated Committees

OF THE

American Society of Civil Engineers
American Society for Testing Materials
American Railway Engineering Association
American Concrete Institute
Portland Cement Association

Submitted to Constituent Organizations

August 14, 1924

PREFACE.

The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete consists of five representatives from each of the following:

American Society of Civil Engineers,
American Society for Testing Materials,
American Railway Engineering Association,
American Concrete Institute,
Portland Cement Association.

This Committee is the successor of the Joint Committee on Concrete and Reinforced Concrete which was organized in Atlantic City, N. J., June 17, 1904, and was formed by the union of special committees appointed in 1903 and 1904 by the above-named organizations, except the American Concrete Institute which was added by invitation of the Joint Committee in 1915. The previous Committee presented progress reports in 1909 and 1912 and adopted a final report to its constituent organizations on July 1, 1916. It was the purpose of that Committee to prepare a Recommended Practice for Concrete and Reinforced Concrete. Its final report stated:

"The report is not a specification but may be used as a basis for specifications."

The present Joint Committee is charged with the preparation of Specifications for Concrete and Reinforced Concrete and in preparing these specifications is using as a basis the report of the former Joint Committee with such modifications as are necessary to make its recommendations agree with current practice, and such new data as mark advances in the art.

The initiative in bringing about the present Joint Committee was taken by the Committee on Reinforced Concrete of the American Society for Testing Materials on June 27, 1917, when the committee voted to request the Executive Committee of the Society to invite the Member-Societies of the previous Joint Committee to cooperate in the formation of a new Joint Committee. The Executive Committee approved this request on April 25, 1919, and an invitation was issued to each of the above-named organizations by the Executive Committee on behalf of the American Society for Testing Materials, to appoint five members on a Joint Committee on Specifications for Reinforced Concrete. The last of these organizations accepted the invitation on November 22, 1919. On January 21,

1920, a call for an organizing meeting on February 11, 1920, was sent by the Executive Committee of that Society to each of the twenty-five representatives of cooperating organizations, together with a list of members of the Joint Committee, and an outline of organization that had been previously submitted by the American Society for Testing Materials to and approved by the cooperating organizations.

The organizing meeting was held at the Engineers' Club, Philadelphia, Pa., and was called to order by George S. Webster, then Vice-President of the American Society for Testing Materials, who explained that he had been directed by the Executive Committee of that Society to act as Temporary Chairman; he further stated that C. L. Warwick, Secretary-Treasurer of the Society, had been requested to act as Temporary Secretary until a formal organization of the Joint Committee had been effected.

The personnel of the Joint Committee is as follows:

American Society of Civil Engineers.

W. A. Slater, *Chairman*,
Engineer-Physicist, Bureau of Standards, Washington, D. C.

Milton H. Freeman, Division Engineer,
N. Y. and N. J. Bridge and Tunnel Commission, New York City.
Appointed June 20, 1922, to fill vacancy.

A. E. Lindau, President, American Wire Fence Co., Chicago, Ill.

Franklin R. McMillan, Consulting Engineer,
628 Metropolitan Bank Building, Minneapolis, Minn.
Appointed June 6, 1921, to fill vacancy.

Sanford E. Thompson, Consulting Engineer,
The Thompson and Lichtner Company, 136 Federal Street, Boston, Mass.

William K. Hatt, Professor of Civil Engineering,
Purdue University, Lafayette, Ind.
Resigned April 3, 1922.

Rudolph P. Miller,
Consulting Engineer, New York City.
Resigned April 25, 1921. Succeeded as Chairman by W. A. Slater.

American Society for Testing Materials.

Richard L. Humphrey, *Chairman*,
Consulting Engineer, Philadelphia, Pa.

Albert T. Goldbeck, Chief, Division of Tests,
Bureau of Public Roads, Washington, D. C.

Edward E. Hughes, Vice-President,
Franklin Steel Works, Franklin, Pa.

Henry H. Quimby, Consulting Engineer,
Philadelphia, Pa.

Leon S. Moisseiff, Engineer of Design,
Delaware River Bridge, Brooklyn, N. Y.

American Railway Engineering Association.[1]

J. J. Yates, *Chairman,*
Bridge Engineer, Central Railroad of New Jersey, Jersey City, N. J.

T. L. D. Hadwen, Engineer of Masonry Construction,
Chicago, Milwaukee and St. Paul Ry., Chicago, Ill.
Appointed November 21, 1921, to fill vacancy.

Frederick E. Schall, Bridge Engineer,
Lehigh Valley Railroad, Bethlehem, Pa.

C. C. Westfall, Engineer of Bridges,
Illinois Central Railroad Company, Chicago, Ill.

George E. Boyd, Formerly Division Engineer,
Delaware, Lackawanna and Western R. R., Buffalo, N. Y.
Resigned November 7, 1921.

H. T. Welty, Engineer of Structures,
New York Central Railroad, New York City.
Resigned December 27, 1922.

American Concrete Institute.[1]

S. C. Hollister, *Chairman,*
Consulting Engineer, Philadelphia, Pa.

Robert W. Lesley, Past-President, Portland Cement Association,
Philadelphia, Pa.

Angus B. MacMillan, Chief Engineer,
Aberthaw Construction Company, Boston, Mass.
Appointed October 19, 1920, to fill vacancy.

Egbert J. Moore, Vice-President,
Turner Construction Company, New York City.

Arthur R. Lord, President,
Lord Engineering Co., Chicago, Ill.
Resigned April 17, 1923.

Leonard C. Wason, President,
Aberthaw Construction Company, Boston, Mass.
Resigned October 19, 1920.

Portland Cement Association.

Frederick W. Kelley, *Chairman,*
President, Portland Cement Association, Albany, N. Y.

Duff A. Abrams, Professor in Charge,
Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

Ernest Ashton, Chemical Engineer,
Lehigh Portland Cement Company, Allentown, Pa.

Edward D. Boyer, Cement Expert,
Atlas Portland Cement Company, New York City.

A. C. Irwin, Engineer, Structural Bureau,
Portland Cement Association, Chicago, Ill.
Appointed September 29, 1922, to fill vacancy.

J. E. Freeman, Formerly Manager, Structural Bureau,
Portland Cement Association, Chicago, Ill.
Appointed January 1, 1921, to fill vacancy.
Resigned September 14, 1922.

J. H. Libberton, Formerly Manager, Service Bureau,
Universal Portland Cement Company, Chicago, Ill.
Resigned December 31, 1920.

[1] At the time of the completion of this report a vacancy existed in the representation of both the American Railway Engineering Association and the American Concrete Institute.

ORGANIZATION OF JOINT COMMITTEE

The Committee perfected a permanent organization on February 11, 1920, under the title "Joint Committee on Standard Specifications for Concrete and Reinforced Concrete" with the following officers:

Chairman, Richard L. Humphrey, Philadelphia, Pa.,
Vice-Chairman, J. J. Yates, Jersey City, N. J.,
Secretary-Treasurer, Duff A. Abrams, Chicago, Ill.,

and an Executive Committee consisting of these officers, and Rudolph P. Miller,[1] New York City, and S. C. Hollister, Philadelphia.

The Committee adopted Rules of Organization and apportioned the work of preparing the Specifications among sub-committees with the following personnel:

1. *Materials (other than Reinforcing).*
   Albert T. Goldbeck, *Chairman*
   Duff A. Abrams
   Sanford E. Thompson
   J. J. Yates
   J. E. Freeman[2]
   J. H. Libberton[3]

2. *Metal Reinforcement.*
   J. J. Yates, *Chairman*
   Duff A. Abrams
   Edward E. Hughes
   A. E. Lindau
   W. K. Hatt[4]
   Arthur R. Lord[5]

3. *Proportioning and Mixing.*
   W. A. Slater, *Chairman*
   Duff A. Abrams
   Ernest Ashton
   T. L. D. Hadwen
   Henry H. Quimby
   George E. Boyd[6]

4. *Forms and Placing*
   T. L. D. Hadwen, *Chairman*
   Edward D. Boyer
   Angus B. MacMillan
   Egbert J. Moore
   Frederick E. Schall
   George E. Boyd,[6] *Formerly Chairman*
   Leonard C. Wason[7]

5. *Design.*
   S. C. Hollister, *Chairman*
   A. C. Irwin
   A. E. Lindau
   Franklin R. McMillan
   Egbert J. Moore
   W. A. Slater
   W. K. Hatt[4]
   Arthur R. Lord[5]
   H. T. Welty[8]

6. *Details of Construction and Fireproofing.*
   Franklin R. McMillan, *Chairman*
   M. H. Freeman
   Leon S. Moisseiff
   C. C. Westfall
   Rudolph P. Miller,[9] *Formerly Chairman*
   W. K. Hatt[4]
   Arthur R. Lord[5]

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[1] Succeeded by W. A. Slater, May 25, 1921.
[2] Resigned September 14, 1922.
[3] Succeeded by J. E. Freeman, January 1, 1921.
[4] Resigned April 3, 1922.
[5] Resigned April 17, 1923.
[6] Succeeded by T. L. D. Hadwen, November 21, 1921.
[7] Succeeded by Angus B. MacMillan, October 19, 1920.
[8] Resigned December 27, 1922.
[9] Succeeded by Franklin R. McMillan, June 6, 1921.

7. *Waterproofing and Protective Treatment.*
Frederick W. Kelley, *Chairman*
Albert T. Goldbeck
S. C. Hollister
Robert W. Lesley
C. C. Westfall

8. *Surface Finish.*
Henry H. Quimby, *Chairman*
Edward D. Boyer
A. C. Irwin
Angus B. MacMillan
J. E. Freeman[1]
J. H. Libberton[2]
Leonard C. Wason[3]
H. T. Welty[4]

9. *Form of Specifications.*
Richard L. Humphrey, *Chairman*
Duff A. Abrams, *Secretary*
T. L. D. Hadwen
Albert T. Goldbeck
S. C. Hollister
Frederick W. Kelley
Franklin R. McMillan
Henry H. Quimby
W. A. Slater
J. J. Yates
George E. Boyd[5]
Rudolph P. Miller[6]

MEETINGS

The Joint Committee held the following meetings:

1920

Organization meeting, Philadelphia, February 11.
Second meeting, Asbury Park, N. J., June 23 and 24.
Third meeting, New York City, October 26, 27, and 28.
Fourth meeting, New York City, December 15, 16, and 17.

1921

Fifth meeting, New York City, March 2, 3, and 4.
Sixth meeting, New York City, April 13, 14, and 15.
Seventh meeting, Asbury Park, N. J., June 21.
Eighth meeting, New York City, October 4 and 5.
Ninth meeting, New York City, December 9.

1922

Tenth meeting, Cleveland, February 17.
Eleventh meeting, Philadelphia, June 1 and 2.
Twelfth meeting, Philadelphia, September 19.
Thirteenth meeting, New York City, October 30 and 31.

[1] Resigned September 14, 1922.
[2] Succeeded by J. E. Freeman, January 1, 1921.
[3] Succeeded by Angus B. MacMillan, October 19, 1920.
[4] Resigned December 27, 1922.
[5] Succeeded by T. L. D. Hadwen, November 21, 1921.
[6] Succeeded by Franklin R. McMillan, June 6, 1921.

1923

Fourteenth meeting, Cincinnati, January 25 and 26.
Fifteenth meeting, New York City, March 28.
Sixteenth meeting, New York City, June 6.
Seventeenth meeting, New York City, October 3, 4, and 5.
Eighteenth meeting, New York City, December 12, 13, and 14.

Numerous meetings of sub-committees were held during the intervals between the meetings of the Joint Committee.

At these meetings the Joint Committee considered the reports of its sub-committees, which were edited by the Sub-Committee on Form and incorporated in the Standard Specifications for Concrete and Reinforced Concrete herewith submitted.

TENTATIVE REPORT SUBMITTED FOR CRITICISM AND DISCUSSION

The Joint Committee submitted to its constituent organizations Tentative Specifications for Concrete and Reinforced Concrete, June 4, 1921. As provided in its Rules of Organization, these Specifications have been open for criticism and discussion by the constituent organizations for a period of more than two years.

These specifications were discussed:

1. At a two-day meeting of the American Society of Civil Engineers, New York City, covering six sessions on December 7 and 8 1921, which discussion was continued for six months in the Society Proceedings;

2. At the Annual Convention of the American Concrete Institute, Cleveland, February 15, 1922;

3. The American Railway Engineering Association referred the Specifications to its Committee on Masonry and the latter discussed them at several of its meetings;

4. The American Society for Testing Materials referred the Specifications to its several committees who were particularly interested and they were discussed at the annual meeting of the Society held at Atlantic City, N. J., June 27–30, 1922.

5. The Portland Cement Association referred them to its Committee on Technical Problems.

FIELD TESTS OF CONCRETE

As a result of the discussion and criticisms the Joint Committee agreed to sponsor a series of field tests made under the auspices of a Joint Committee of Contractors, on which the Associated General Contractors of America were represented, with the object of deter-

mining whether the recommendations of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete were practical, especially as regards the provisions for the control of the quality of the concrete in the field.

Tests were made during the summer of 1923 on the concrete used in the construction of Building No. 10 of the Victor Talking Machine Company, Camden, N. J., through the cooperation of the owner and of Stone and Webster, Inc., the contractors; these tests have been completed and with a discussion of the results will be available in the near future.

A second series of tests were made of the concrete being used in the construction of the piers of the Newark Bay Bridge of the Central Railroad of New Jersey through the cooperation of the Railroad Company; these tests are practically completed; it is expected that other tests will be made during the present year.

All of these tests have been made possible through the cooperation of the owners of the structures at which the tests were made, the U. S. Department of Commerce (Bureau of Standards), Department of Highways of the Commonwealth of Pennsylvania, the Joint Committee of Contractors, and the Structural Materials Research Laboratory, Lewis Institute, to whom the thanks and appreciation of the Joint Committee are extended.[1]

The results thus far obtained are a substantial justification of the Joint Committee's recommendations.

Another series of tests is to be inaugurated under the auspices of the Joint Committee on Concrete Reinforcement of the American Society for Testing Materials, having as an object the development of data as a basis for specifications for metal reinforcement.

STANDARD SPECIFICATIONS

The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete has carefully reviewed the criticisms and discussions of its recommendations and the data that have become available since it submitted the Tentative Specifications in 1921, and has made such modifications in the Specifications as were desirable.

The Committee is of the opinion that it should now submit the Standard Specifications for Concrete and Reinforced Concrete; with the later data as the result of tests now in progress it may be deemed desirable to submit revisions in the light of the information obtained.

The Committee in submitting the Specifications directs attention

[1] For the Report on the Camden and Newark Bay Tests, see *Proceedings*, Am. Soc. Civil Engrs., December, 1924.

to the fact that while the Specifications are complete as regards the general uses of concrete and reinforced concrete, they may require some additional paragraphs to cover the use of these materials for special purposes.

The Committee has under consideration supplementary specifications covering special uses of concrete and reinforced concrete on which it is not prepared to report at this time.

The Joint Committee further calls attention to the fact that it has undertaken to prepare specifications covering the fundamentals to be observed in the general use of concrete and reinforced concrete; no attempt has been made to cover the details involved in the use of these materials in special structures. Although the sections relating to design deal primarily with building construction, nevertheless the principles involved are of general application to structures of other types.

This report has been submitted to letter ballot of the Joint Committee which consists of 23 members,[1] representing the five societies, 22 of whom have voted affirmatively, none negatively, and one has refrained from voting.

Respectfully submitted,

RICHARD L. HUMPHREY, *Chairman.*

J. J. YATES, *Vice-Chairman*
ERNEST ASHTON
EDWARD D. BOYER
MILTON H. FREEMAN
ALBERT T. GOLDBECK
T. L. D. HADWEN
S. C. HOLLISTER
EDWARD E. HUGHES
A. C. IRWIN
FREDERICK W. KELLEY
ROBERT W. LESLEY
DUFF A. ABRAMS, *Secretary-Treasurer*
A. E. LINDAU
ANGUS B. MACMILLAN
FRANKLIN R. MCMILLAN
LEON S. MOISSEIFF
EGBERT J. MOORE
HENRY H. QUIMBY
FREDERICK E. SCHALL
W. A. SLATER
SANFORD E. THOMPSON

[1] At the time of the completion of this report a vacancy existed in the representation of both the American Railway Engineering Association and the American Concrete Institute.

STANDARD SPECIFICATIONS

FOR

CONCRETE AND REINFORCED CONCRETE

Submitted by the

JOINT COMMITTEE ON STANDARD SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

Affiliated Committees

OF THE

American Society of Civil Engineers
American Society for Testing Materials
American Railway Engineering Association
American Concrete Institute
Portland Cement Association

August 14, 1924

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LIST OF APPENDICES

The Mandatory Appendices 1 to 15, inclusive, consisting of Notation, Figures, Specifications and Methods of Tests, form a part of the Standard Specifications for Concrete and Reinforced Concrete. Advisory Appendices 16, 17 and 18 are for information only.

Mandatory Appendices[1] PAGE.

Advisory Appendices

[1] The Standard and Tentative Specifications and Methods of Tests are those of the American Society for Testing Materials, 1315 Spruce St., Philadelphia, Pa. See Book of A.S.T.M. Standards for 1924; also Tentative Standards for 1924.

STANDARD SPECIFICATIONS

FOR

CONCRETE AND REINFORCED CONCRETE.

CHAPTER I. GENERAL INSTRUCTIONS.

1. These Specifications cover the general conditions affecting the use of concrete and reinforced concrete. To use them in connection with the contract it will be necessary for the Engineer to: **General Instructions.**

(*a*) Provide the detail specifications covering the work in particular in which the concrete and reinforced concrete are to be used;

(*b*) Section 7, Table I, insert percentages required;

(*c*) Section 10, fill blank indicating percentage of mortar strength required of fine aggregate as compared with standard Ottawa sand;

(*d*) Section 14, Table II, insert sieve sizes and percentages required;

(*e*) Section 21, strike out one of the titles of specifications for reinforcement bars; if billet-steel bars are to be used, indicate whether structural, intermediate or hard grade.

(*f*) Section 24, strike out one of the titles of specifications for structural steel shapes;

(*g*) Section 28, strike out "volume" or "weight;"

(*h*) Section 29, if no tests of concrete are to be made, strike out last paragraph in accordance with footnote. Table IV, insert the concrete strengths which are assumed as basis for design of the different portions of the work;

(*i*) Section 30, Table V, insert the slumps required;

(*j*) Section 51, strike out the method or methods for depositing concrete under water which are not applicable to the work;

(*k*) Chapter X, strike out sections on surface finish which do not apply;

(*l*) Section 95, strike out one of the two sections on terrazzo finish.

CHAPTER II. DEFINITIONS.

2. The following definitions give the meaning of certain terms as used in these specifications: **Definitions.**

Acid Proofing.—Treatment of a concrete surface in order to provide resistance to the action of acids.

Aggregate.—Inert material which is mixed with portland cement and water to produce concrete; in general, aggregate consists of sand,

pebbles, gravel, crushed stone, or similar materials. (See *Fine Aggregate, Coarse Aggregate.*)

Anchorage.—The embedment in concrete of a portion of a reinforcement bar, either straight or with hooks, designed to prevent pulling out or slipping of the bar when subjected to stress. (The anchorage of tension reinforcement in beams includes only the embedded length beyond a point of contra-flexure or of zero moment.)

Approved.—Meeting the approval of, or specifically authorized by, the Engineer.

Buttressed Retaining Wall.—A retaining wall with brackets or buttresses on the side opposite the pressure face uniting the upright section with the toe of the base.

Cantilever Retaining Wall.—A reinforced concrete wall having an upright section and a base, each of which resists by cantilever action the pressure to which it is subjected.

Cellular Retaining Wall.—A retaining wall with a base, longitudinal upright sections, and a series of transverse walls, dividing the space between the longitudinal sections into cells which are filled with earth or other suitable material.

Coarse Aggregate.—Aggregate, subject to specified tolerances, retained on a No. 4 sieve and of a maximum size generally not larger than 3 in. (See *Aggregate, Fine Aggregate.*)

Column.—An upright compression member the length of which exceeds three times its least lateral dimension.

Column Capital.—An enlargement of the upper end of a reinforced concrete column designed and built to act as a unit with the column and flat slab.

Column Strip.—A portion of a panel of a flat slab which has a uniform width equal to one-fourth of the panel length perpendicular to the direction of the strip, and the outer edge of which lies on the edges of the panel. (See *Middle Strip;* also Appendix 1, Fig. 15.)

Composite Column.—A circumferentially reinforced concrete column with a core of structural steel or cast iron which is designed to carry a portion of the load.

Concrete.—A mixture of portland cement, fine aggregate, coarse aggregate and water. (See *Mortar.*)

Consistency.—A general term used to designate the relative plasticity of freshly mixed concrete or mortar.

Counterforted Retaining Wall.—A reinforced concrete wall with brackets or counterforts on the pressure face uniting the upright section to the heel of the base.

Crusher-Run Stone.—Unscreened crushed stone. (See *Stone Screenings.*

Cyclopean Concrete.—Concrete in which stones weighing more than 100 lb. are individually embedded.

Dead Load.—The weight of the permanent parts of the structure.

Deformed Bar.—Reinforcement bar with shoulders, lugs, or projections formed integrally with the bar during rolling.

Diagonal Direction.—A direction parallel or approximately parallel to the diagonal of the panel of a flat slab.

Dropped Panel.—The structural portion of a flat slab which is thickened throughout an area surrounding the column capital.

Effective Area of Concrete.—The area of a section of the concrete which lies between the tension reinforcement and the compression surface in a beam or slab.

Effective Area of Reinforcement.—The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between its direction and that for which the effectiveness of the reinforcement is to be determined.

Engineer.—The engineer in responsible charge of the work.

Fine Aggregate.—Aggregate, subject to specified tolerances, passing through a No. 4 sieve. (See *Aggregate, Coarse Aggregate.*)

Flat Slab.—A concrete slab having reinforcement bars extending in two or more directions without beams or girders to carry the load to supporting members.

Footing.—A structural unit used to distribute wall or column loads to the foundation materials.

Gravel.—Rounded particles larger than sand resulting from the natural disintegration of rocks. (See *Sand.*)

Laitance.—Extremely fine material of little or no hardness which may collect on the surface of freshly-deposited concrete or mortar, resulting from the use of excess mixing water, and usually recognized by its relatively light color.

Live Load.—Loads and forces other than the dead load.

Middle Strip.—The portion of a panel of a flat slab which extends in a direction parallel to a side of the panel, the width of which is one-half the panel length at right angles to the direction of the strip and whose center line lies on the center line of the panel. (See *Column Strip;* also Appendix 1, Fig. 15.)

Mortar.—A mixture of portland cement, fine aggregate and water. (See *Concrete.*)

Negative Reinforcement.—Reinforcement so placed as to take tensile stress due to negative bending moment.

Oilproofing.—Treatment of a concrete surface for the purpose of preventing the penetration of, and resisting the action of, oils.

Panel Length.—The distance in either rectangular direction between centers of two columns of a panel.

Pedestal.—An upright compression member whose height does not exceed three times its least lateral dimension.

Pedestal Footing.—A column footing projecting less than one-half its depth from the faces of the column on all sides and having a depth not more than three times its least width.

Plain Concrete.—Concrete without metal reinforcement.

Portland Cement.—The product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

Positive Reinforcement.—Reinforcement so placed as to take tensile stress due to positive bending moment.

Principal Design Section.—The vertical sections in a flat slab on which the moments in the rectangular directions are critical. (See Section 143.)

Ratio of Reinforcement.—The ratio of the effective area of the reinforcement cut by a section of a beam or slab to the effective area of the concrete cut by that section.

Rectangular Direction.—A direction parallel to a side of the panel of a flat slab.

Reinforced Concrete.—Concrete in which metal is embedded in such a manner that the two materials act together in resisting forces.

Rubble Aggregate.—Stone or gravel larger than 3 in. in diameter and weighing not more than 100 lb.

Rubble Concrete.—Concrete in which pieces of rubble aggregate are individually embedded. (See *Rubble Aggregate.*)

Sand.—Small grains resulting from the natural disintegration of rocks. (See *Gravel.*)

Screen.—A metal plate with closely spaced circular perforations. (See *Sieve.*)

Sieve.—Woven wire cloth with square openings. (See *Screen.*)

Slump.—The shortening of a standard test mass of freshly mixed concrete, used as a measure of workability in accordance with the standard method. (See Appendix 11.)

Standard Sand.—Natural sand from Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve, used as the fine aggregate in standard strength tests of portland cement. (For specifications see Appendix 2.)

Stone Screenings.—Crushed stone, subject to specified tolerances, passing through a No. 4 sieve. (See *Crusher-Run Stone.*)

Strut.—A compression member other than a column or pedestal.

Tremie.—A water-tight pipe of suitable dimensions, generally used in a vertical position, for depositing concrete under water.

Wall Beam.—A reinforced concrete beam which extends from column to column along the outer edge of a wall panel.

CHAPTER III. QUALITY OF CONCRETE

3. The design of the various portions of the structure is based on the assumption that the concrete will develop the compressive strength at 28 days which, for purposes of design only, are given in Section 29, Table IV. **Strength.**

4. The making, curing and testing of field concrete test specimens shall be carried out in accordance with the Standard Methods of Making and Storing Specimens of Concrete in the Field (Serial Designation: C 31 – 21) of the American Society for Testing Materials. (Appendix 13.) **Tests of Field Specimens.**

CHAPTER IV. MATERIALS.[1]

A. *Portland Cement.*

5. Portland cement shall conform to the Standard Specifications and Tests for Portland Cement (Serial Designation: C 9 – 21) of the American Society for Testing Materials[2] (Appendix 2) and subsequent revisions thereof. **Portland Cement.**

B. *Fine Aggregate.*

6. Fine aggregate shall consist of sand or other approved inert materials with similar characteristics, or a combination thereof, having clean, hard, strong, durable, uncoated grains and free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter, loam or other deleterious substances. **General Requirements.**

7. Fine aggregate shall range in size from fine to coarse within the limits indicated in Table I. **Grading.**

TABLE I.—GRADING OF FINE AGGREGATE.

	PER CENT BY WEIGHT
Passing through No. 4 sieve	not less than (*a*)[3]
Passing through No. 50 sieve	not more than (*b*)[3] not less than (*c*)[3]
Weight removed by decantation	not more than (*d*)[3]

[1] These Specifications cover the usual requirements for aggregate. Where concrete having special properties is essential the specifications should make clear the characteristics of the aggregate to be used.

[2] These Specifications are also the standard of the following: United States Government, American Railway Engineering Association, American Concrete Institute, and Portland Cement Association. Approved by the American Engineering Standards Committee as an American Standard.

[3] The grading requirements will vary with the type of work and characteristics of materials available in any given locality. Wherever practicable the following values should be inserted: (*a*) = 85 per cent; (*b*) = 30 per cent; (*c*) = 10 per cent; (*d*) = 3 per cent.

Sieve Analysis. 8. The sieves and method of making sieve analysis shall conform to the Standard Method of Test for Sieve Analysis of Aggregates for Concrete (Serial Designation: C 41 – 24) of the American Society for Testing Materials. (Appendix 8.)

Decantation Test. 9. The decantation test shall be made in accordance with the Tentative Method of Decantation Test for Sand and Other Fine Aggregates (Serial Designation: D 136 – 22 T) of the American Society for Testing Materials. (Appendix 9.)

Mortar Strength Test. 10. Fine aggregate shall be of such quality that mortar briquettes, cylinders or prisms, consisting of one part by weight of portland cement and three parts by weight of fine aggregate,[1] mixed and tested in accordance with the methods described in the Standard Specifications and Tests for Portland Cement (Appendix 2) will show a tensile or compressive strength at ages of 7 and 28 days not less than ...... per cent[2] of that of 1:3 standard Ottawa sand mortar of the same plasticity made with the same cement. Concrete tests shall be made in accordance with the Tentative Methods of Making Compression Tests of Concrete (Serial Designation: C 39 – 21 T) of the American Society for Testing Materials. (Appendix 12.)

Organic Impurities in Sand. 11. Sand, when tested in accordance with the Standard Method of Test for Organic Impurities in Sand for Concrete (Serial Designation: C 40 – 22) of the American Society for Testing Materials (Appendix 10), shall show a color not darker than the standard color unless it complies with Section 10.

Permissible Variations. 12. Fine aggregate which does not conform to the above requirements for grading, mortar strength or color, may be used only when approved by the Engineer and then in such proportions as he may require.

C. *Coarse Aggregate.*[3]

General Requirements. 13. Coarse aggregate shall consist of crushed stone, gravel, or other approved inert materials with similar characteristics, or combinations thereof, having clean, hard, strong, durable, uncoated particles free from injurious amounts of soft, friable, thin, elongated or laminated pieces, alkali, organic or other deleterious matter.

Grading. 14. Coarse aggregate[4] shall range in size from fine to coarse within the limits given in Table II.

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[1] In testing aggregate, care should be exercised to avoid the removal of any coating on the grains which may affect the strength. Sand should not be dried before being made into mortar, but should contain its natural moisture. The quantity of water contained may be determined on a separate sample and the weight of the sand used in the test corrected accordingly.

[2] This percentage must be inserted by the Engineer; it should preferably be 100.

[3] Appendix 16 furnishes a guide in determining the proportions of materials required to produce concrete of a given strength, using aggregates of different sizes and concrete of different consistencies.

[4] On work of considerable magnitude where several suitable coarse aggregates are available, an investigation of the relative economy of each is advisable.

TABLE II.—SIZE AND GRADING OF COARSE AGGREGATE.

Passing		Per Cent by Weight
———[a] in. sieve (maximum size)		not less than 95
———[a] in. " (intermediate size)		not less than ———[a] not more than ———[a]
No. 4	"	not more than 10
No. 8	"	not more than 5

[a] The Engineer must insert in these blanks the sizes and percentages required with regard to materials available. The following table indicates desirable gradings for coarse aggregates of certain nominal maximum sizes:

Nominal Maximum Size of Aggregate, in.	Percentage by Weight Passing through Standard Sieves with Square Openings						Percentage Passing, not more than	
	3 in.	2 in.	$1\frac{1}{2}$ in.	1 in.	$\frac{3}{4}$ in.	$\frac{1}{2}$ in.	No. 4 Sieve	No. 8 Sieve
3	95	..	40–75	..	..	..	10	5
2	..	95	..	40–75	..	..	10	5
$1\frac{1}{2}$	..	..	95	..	40–75	..	10	5
1	..	..	..	95	..	..	10	5
$\frac{3}{4}$	..	..	..	..	95	..	10	5
$\frac{1}{2}$	..	..	..	..	..	95	10	5

15. The test for size and grading of aggregate shall be made in accordance with the Standard Method of Test for Sieve Analysis of Aggregates for Concrete. (Appendix 8.) **Sieve Sizes.**

16. Coarse aggregate[1] which does not conform to the above requirements may be used only when approved by the Engineer and then in such proportions as he may require. **Permissible Variations.**

D. Rubble and Cyclopean Aggregate.

17. Rubble aggregate shall consist of clean, hard, durable stone or gravel larger than 3 in. and weighing not more than 100 lb. **Rubble Aggregate.**

18. Cyclopean aggregate shall consist of clean, hard, durable stone or gravel weighing more than 100 lb. **Cyclopean Aggregate.**

E. Storage of Aggregate.

19. Aggregate shall be so stored as to avoid the inclusion of foreign materials. Frozen aggregate or aggregate containing lumps of frozen material shall be thawed before using. **Storage of Aggregate.**

F. Water.

20. Water for concrete shall be clean and free from injurious amounts of oil, acid, alkali, organic matter, or other deleterious substance. **General Requirements.**

[1] Requirements for the quality of coarse aggregate for special purposes should be inserted.

G. Metal Reinforcement.

Quality. 21. Metal reinforcement shall conform to the requirements of the Standard Specifications[1] for Billet-Steel Concrete Reinforcement Bars ...... grade (Serial Designation: A 15 – 14) of the American Society for Testing Materials (Appendix 3), Standard Specifications[1] for Rail-Steel Concrete Reinforcement Bars (Serial Designation: A 16 – 14) of the American Society for Testing Materials (Appendix 4), except that the provision for machining deformed bars before testing shall be eliminated.

Standard Sizes of Bars. 22. Reinforcement bars shall conform to the areas and equivalent sizes shown in Table III.

TABLE III.—SIZES AND AREAS OF REINFORCEMENT BARS.

Size of Bar, in.	Area, sq. in.	
	Round Bar	Square Bar
$\frac{3}{8}$	0.110	.....
$\frac{1}{2}$	0.196	0.250
$\frac{5}{8}$	0.306	.....
$\frac{3}{4}$	0.441	.....
$\frac{7}{8}$	0.601	.....
1	0.785	1.000
$1\frac{1}{8}$	.....	1.265
$1\frac{1}{4}$	.....	1.562

Deformed Bars. 23. An approved deformed bar shall be one that will develop a bond at least 25 per cent greater than that of a plain round bar of equivalent cross-sectional area.[2] The areas of deformed bars shall be determined by the minimum cross-section thereof.

Structural Shapes. 24. Structural steel shapes used for reinforcement shall conform to the requirements of the Standard Specifications[3] for Structural Steel for Bridges (Serial Designation: A 7 – 24) of the American Society for Testing Materials (Appendix 5), Standard Specifications[3] for Structural Steel for Buildings (Serial Designation: A 9 – 24) of the American Society for Testing Materials. (Appendix 6.)

Wire. 25. Wire for concrete reinforcement shall conform to the requirements of the Tentative Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement (Serial Designation: A 82 – 21 T) of the American Society for Testing Materials. (Appendix 7.)

[1] The engineer must strike out one of these titles. The Committee recommends as preferred material for reinforcement that meeting the requirements of the Standard Specifications for Billet-Steel Concrete Reinforcement Bars of intermediate grade, made by the open-hearth process, except that the provision for machining the test specimens shall be eliminated.

[2] The Committee has under consideration a specification for deformed bars but is not prepared at this time to make more definite recommendations.

[3] The engineer must strike out one of these titles.

26. Cast iron used in composite columns shall conform to the requirements of the Standard Specifications for Cast-Iron Pipe and Special Castings (Serial Designation: A 44 – 04) of the American Society for Testing Materials. (Appendix 14.) **Cast Iron.**

CHAPTER V. PROPORTIONING AND MIXING CONCRETE

A. *Proportioning.*

27. The unit of measure shall be the cubic foot. Ninety-four pounds of cement (one bag or $\frac{1}{4}$ bbl.) shall be considered as one cubic foot. **Unit of Measure.**

28. The method of measurement shall be such as to secure the specified proportions in each batch. The aggregates shall be measured **Measurement of Aggregates.**

TABLE IV.—PROPORTIONS FOR CONCRETE.[a]

Part of Structure	Proportions by Volume			Concrete Strength[b] Assumed as Basis for Design, lb. per sq. in at 28 Days
	Cement	Fine Aggregate	Coarse Aggregate	
Footings	..	..	..	..[c]
Beams, Girders, Slabs	..	..	..	..
Columns	..	..	..	..
....	..	..	..	..
....	..	..	..	..
....	..	..	..	..

[a] The Engineer should determine, by tests of the available aggregate in advance of use, the proportions necessary to produce concrete of the required strength. Where this is impracticable Appendix 16 may be used as a guide. Appendix 16 is based on volumes of dry aggregate compacted by rodding in the measure, as specified in the Standard Method of Test for Unit Weight of Aggregate for Concrete (Serial Designation: C 29 – 21) of the American Society for Testing Materials. (See Appendix 15.) Corrections should be made in the quantities in Appendix 16 to take account of the bulking effect of moisture in the fine aggregate. The bulking of fine aggregate (swelling) due to contained moisture, and the method of placing it in the measure, may result in a reduction of 25 per cent in the actual quantity of fine aggregate, as compared with that obtained by dry measurement by the standard method.

[b] The measure of the quality of concrete is its workability, as determined by the slump test or other approved test, and its compressive strength at 28 days, using the materials in the proportions specified.

[c] Concrete strengths to be used as basis for design shall be filled in by the Engineer.

separately by volume,[1] weight.[1] In volume measurement the fine aggregate and the coarse aggregate shall be measured loose, as thrown into the measuring device, and struck off.[2] The water shall be so measured as to insure the desired quantity in successive batches.

29. Concrete shall be mixed in the proportions indicated in Table IV. **Proportions.**

Variations in the grading of the aggregates, on which the proportions were based, may be made upon the approval of the Engineer and in such proportions as he may direct (Sections 3 and 14), but no claim shall be made for extra compensation therefor.

[1] The Engineer must strike out one of these terms.

[2] The volume of fine aggregate is affected by the method of measurement and by the moisture content; corrections should be made, when necessary, to maintain the required proportions. See also footnote to Table IV.

The Engineer shall have the right to make any changes in proportions or materials that may be necessary or desirable, and proper adjustment of compensation will be made in accordance with the provisions of the contract.

In general the quantity of water used shall be the minimum necessary to produce concrete of a workability specified in Table V.

TABLE V.—WORKABILITY OF CONCRETE.

Type of Concrete.	Maximum Slump, in.
1. Mass concrete	..[a]
2. Reinforced concrete:	
(a) Thin vertical sections and columns	..[a]
(b) Heavy sections	..[a]
(c) Thin confined horizontal sections	..[a]
3. Roads and pavements:	
(a) Hand finished	..[a]
(b) Machine finished	..[a]
4. Mortar for floor finish	..[a]

[a] The engineer must insert the maximum slumps permitted. The slump test requirement is intended to insure concrete mixed with the minimum quantity of water required to produce a plastic mixture. The following table indicates the maximum slump desirable for the various types of concrete, based on average aggregates and proportions:

Type of Concrete.	Maximum Slump, in.
1. Mass concrete	3
2. Reinforced concrete:	
(a) Thin vertical sections and columns	6
(b) Heavy sections	3
(c) Thin confined horizontal sections	8
3. Roads and pavements:	
(a) Hand finished	3
(b) Machine finished	1
4. Mortar for floor finish	2

Frequent tests[1] shall be made throughout the work, as directed by the Engineer, to determine whether the concrete produced by the proportions given in Table IV is of the quality specified. These tests shall be made in accordance with Section 4 and at the expense of the owner. The contractor shall provide such facilities as may be practicable for carrying out the tests, and shall cooperate in every way to the end that concrete of the desired quality shall be obtained.

B. Consistency of Concrete.

Consistency. 30. The quantity of water used shall be the minimum necessary to produce concrete of a workability required by the Engineer.[2] The consistency of the concrete shall be measured by the slump test as

[1] If no tests are to be made on concrete from available materials, or on the concrete mixed during the work, this paragraph shall be omitted.

[2] Attention is called to the fact that increased workability may be obtained by decreasing the quantity of coarse aggregate in the batch, without increasing the quantity of mixing water.

described in the Tentative Method of Test for Consistency of Portland-Cement Concrete for Pavements or for Pavement Base (Serial Designation: D 138 – 22 T) of the American Society for Testing Materials. (Appendix 11.) The slump for the different types of concrete shall not be greater than that indicated in Table V, unless authorized by the Engineer.

The consistency shall be checked from time to time during the progress of the work.

C. *Mixing.*

31. The mixing of concrete, unless otherwise authorized by the Engineer, shall be done in a batch mixer of approved type which will insure a uniform distribution of the materials throughout the mass, so that the mixture is uniform in color and homogeneous. The mixer shall be equipped with suitable charging hopper, water storage, and a water-measuring device controlled from a case which can be kept locked and so constructed that the water can be discharged only while the mixer is being charged. It shall also be equipped with an attachment for automatically locking the discharge lever until the batch has been mixed the required time after all materials are in the mixer. The entire contents of the drum shall be discharged before recharging. The mixer shall be cleaned at frequent intervals while in use. The volume of the mixed material per batch shall not exceed the manufacturer's rated capacity of the mixer. **Machine Mixing.**

32. The mixing of each batch shall continue not less than one minute after all the materials are in the mixer, during which time the mixer shall rotate at a peripheral speed of about 200 ft. per minute. **Time of Mixing.**

33. When hand mixing is authorized by the Engineer it shall be done on a water-tight platform. The cement and fine aggregate shall first be mixed dry until the whole is of a uniform color. The water and coarse aggregate shall then be added and the entire mass turned at least three times, or until a homogeneous mixture of the required consistency is obtained. **Hand Mixing.**

34. The retempering of concrete or mortar which has partially hardened, that is, remixing with or without additional cement, aggregate, or water, will not be permitted. **Retempering.**

CHAPTER VI. DEPOSITING CONCRETE.

A. *Depositing in Air.*

35. Before beginning a run of concrete, hardened concrete and foreign materials shall be removed from the inner surfaces of the mixing and conveying equipment. **General.**

Approval. 36. Before depositing concrete, debris shall be removed from the space to be occupied by the concrete; forms shall be thoroughly wetted (except in freezing weather) or oiled. Reinforcement shall be thoroughly secured in position and approved by the Engineer.

Handling. 37. Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable by methods which shall prevent the separation or loss of the ingredients. It shall be deposited in the forms as nearly as practicable in its final position to avoid rehandling. It shall be so deposited as to maintain, until the completion of the unit, a plastic surface approximately horizontal. Forms for walls or other thin sections of considerable height, shall be provided with openings, or other devices, that will permit the concrete to be placed in a manner that will avoid accumulations of hardened concrete on the forms or metal reinforcement. Under no circumstances shall concrete that has partially hardened be deposited in the work.

Chuting. 38. When concrete is conveyed by chuting, the plant shall be of such size and design as to insure a practically continuous flow in the chute. The angle of the chute with the horizontal shall be such as to allow the concrete to flow without separation of the ingredients.[1] The delivery end of the chute shall be as close as possible to the point of deposit. When the operation is intermittent, the spout shall discharge into a hopper. The chute shall be thoroughly flushed with water before and after each run; the water used for this purpose shall be discharged outside the forms.

Compacting. 39. Concrete, during and immediately after depositing, shall be thoroughly compacted by means of suitable tools. For thin walls or inaccessible portions of the forms, where rodding or forking is impracticable, the concrete shall be assisted into place by tapping or hammering the forms opposite the freshly deposited concrete. The concrete shall be thoroughly worked around the reinforcement, and around embedded fixtures, and into the corners of the forms.

Removal of Water. 40. Water shall be removed from excavations before concrete is deposited, unless otherwise directed by the Engineer. Any flow of water into the excavation shall be diverted through proper side drains to a sump, or be removed by other approved methods which will avoid washing the freshly deposited concrete. Water vent pipes and drains shall be filled by grouting or otherwise, after the concrete has thoroughly hardened.

Protection. 41. Exposed surfaces of concrete shall be protected from premature drying for a period of at least seven days after being deposited.

[1] An angle of 27 deg., or one vertical to two horizontal, is the minimum slope which is considered permissible. Chuting through a vertical pipe is satisfactory when the lower end of the pipe is maintained as nearly as practicable to the surface of deposit, and the pipe full.

42. Concrete when deposited shall have a temperature of not less than 40° F. nor more than 120° F. In freezing weather suitable means shall be provided for maintaining the concrete at a temperature of at least 50° F. for not less than 72 hours after placing, or until the concrete has thoroughly hardened. The methods of heating the materials and protecting the concrete shall be approved by the Engineer. Salt, chemicals, or other foreign materials shall not be mixed with the concrete for the purpose of preventing freezing, unless approved by the Engineer. **Temperature of Concrete.**

43. Concrete shall be deposited continuously and as rapidly as practicable until the unit of operation, approved by the Engineer, is completed. Construction joints at points not provided for in the plans shall be made in accordance with the provisions in Section 69. **Depositing Continuously.**

44. Before depositing new concrete on or against concrete which has set, the forms shall be retightened, the surface of the set concrete shall be roughened as required by the Engineer, thoroughly cleaned of foreign matter and laitance, and saturated with water. The new concrete placed in contact with hardened or partially hardened concrete, shall contain an excess of mortar to insure bond. To insure this excess mortar at the juncture of the hardened and the newly deposited concrete, the cleaned and saturated surfaces of the hardened concrete, including vertical and inclined surfaces, shall first be slushed with a coating of neat cement grout against which the new concrete shall be placed before the grout has attained its initial set. **Bonding.**

B. Rubble and Cyclopean Concrete.

45. Rubble aggregate shall be thoroughly embedded in the concrete. The individual stones shall not be closer to any surface or adjacent stone than the maximum size of the coarse aggregate plus 1 in. Each successive layer of concrete shall be keyed in accordance with the provision in Section 69. **Rubble Concrete.**

46. Cyclopean aggregate shall be thoroughly embedded in the concrete; no stone shall be closer to a finished surface than 1 ft., nor closer than 6 in. to any adjacent stone. Stratified stone shall be laid on its natural bed. **Cyclopean Concrete.**

C. Depositing Under Water.[1]

47. The methods, equipment, and materials to be used shall be submitted to and approved by the Engineer before the work is started. **General.**

[1] Concrete should not be deposited under water if practicable to deposit in air. There is always uncertainty as to the results obtained from placing concrete under water. Where conditions permit, the additional expense and delay of avoiding this method will be warranted. It is especially important that the aggregate be free from loam and other material which may cause laitance. Washed aggregates are preferable. Coarse aggregate consisting of washed gravel of a somewhat smaller size than that used in open-air concrete work will give best results. Concrete should never be deposited under water without experienced supervision. Many failures, especially of structures in sea water, can be traced directly to ignorance of proper methods or lack of expert supervision.

Concrete shall be deposited by a method that will prevent the washing of the cement from the mixture, minimize the formation of laitance and avoid flow of water until the concrete has fully hardened. Concrete shall be placed so as to minimize segregation of materials. Concrete shall not be placed in water having a temperature below 35° F.

Proportions. 48. Concrete to be deposited under water shall contain $1\frac{3}{4}$ bbl. (7 bags) or more of portland cement per cubic yard of concrete in place.

Cofferdams. 49. Cofferdams shall be sufficiently tight to prevent flow of water through the space in which the concrete is to be deposited. Pumping will not be permitted while concrete is being deposited, nor until it has fully hardened.

Depositing Continuously. 50. Concrete shall be deposited continuously, keeping the top surface as nearly level as possible, until it is brought above the water, or to the required height. The work shall be carried on with sufficient rapidity to prevent the formation of layers.

Method. 51. The following method[1] shall be used for depositing concrete under water:

(*a*) *Tremie.*—The tremie shall be water-tight and sufficiently large to permit a free flow of concrete. It shall be kept filled[2] at all times during depositing. The concrete shall be discharged and spread by raising the tremie in such manner as to maintain as nearly as practicable a uniform flow and avoid dropping the concrete through water. If the charge is lost during depositing the tremie shall be withdrawn and refilled.

(*b*) *Drop-Bottom Bucket.*—The bucket shall be of a type that cannot be dumped until it rests on the surface upon which the concrete is to be deposited. The bottom doors when tripped shall open freely downward and outward. The top of the bucket shall be open. The bucket shall be completely filled, and slowly lowered to avoid back-wash. When discharged, the bucket shall be withdrawn slowly until well above the concrete.

(*c*) *Bags.*—Bags of jute or other coarse cloth shall be filled about two-thirds full of concrete and carefully placed by hand in a header-and-stretcher system so that the whole mass is interlocked.

Laitance. 52. Great care shall be exercised to disturb the concrete as little as possible when it is being deposited in order to avoid the formation of laitance. On completing a section of concrete, the laitance shall be entirely removed before work is resumed.

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[1] The engineer must strike out the method or methods not applicable to the work.

[2] The tremie may be filled by one of the following methods: (1) Place the lower end in a box partly filled with concrete, so as to seal the bottom, then lower into position; (2) plug the tremie with cloth sacks or other material, which will be forced down as the pipe is filled with concrete; (3) plug the end of the tremie with cloth sacks filled with concrete.

CHAPTER VII. FORMS.

53. Forms shall conform to the shape, lines and dimensions of the concrete as called for on the plans. Lumber used in forms for exposed surfaces shall be dressed to a uniform thickness, and shall be free from loose knots or other defects. Joints in forms shall be horizontal or vertical. For unexposed surfaces and rough work, undressed lumber may be used. Lumber once used in forms shall have nails withdrawn, and surfaces to be in contact with concrete thoroughly cleaned, before being used again. **General.**

54. Forms shall be substantial and sufficiently tight to prevent leakage of mortar; they shall be properly braced or tied together so as to maintain position and shape. If adequate foundation for shores cannot be secured, trussed supports shall be provided. **Design.**

55. Bolts and rods shall preferably be used for internal ties; they shall be so arranged that when the forms are removed no metal shall be within 1 in. of any surface. Wire ties will be permitted only on light and unimportant work; they shall not be used through surfaces where discoloration would be objectionable. Shores supporting successive stories shall be placed directly over those below, or so designed that the load will be transmitted directly to them. Forms shall be set to line and grade and so constructed and fastened as to produce true lines. Special care shall be used to prevent bulging. **Workmanship.**

56. Unless otherwise specified, suitable moldings or bevels shall be placed in the angles of forms to round or bevel the edges of the concrete. **Moldings.**

57. The inside of forms shall be coated with non-staining mineral oil or other approved material or thoroughly wetted (except in freezing weather). Where oil is used, it shall be applied before the reinforcement is placed. **Oiling.**

58. Temporary openings shall be provided at the base of column and wall forms, and at other points where necessary to facilitate cleaning and inspection immediately before depositing concrete. **Inspection.**

59. Forms shall not be disturbed until the concrete has adequately hardened.[1] Shoring shall not be removed until the member has acquired sufficient strength to safely support its weight and the load upon it. Members subject to additional loads during construction shall be adequately shored to support both the member and construction loads in such a manner as will protect the member from damage by the loads; this shoring shall not be removed until the member has acquired sufficient strength to safely support its weight and the load upon it. **Removal.**

[1] Many conditions affect the hardening of concrete and the proper time for the removal of the forms should be determined by the Engineer.

CHAPTER VIII. DETAILS OF CONSTRUCTION.

A. Metal Reinforcement.

Cleaning. 60. Metal reinforcement, before being positioned, shall be thoroughly cleaned of mill and rust scale and of coatings that will destroy or reduce the bond. Reinforcement appreciably reduced in section shall be rejected. Where there is delay in depositing concrete, reinforcement shall be re-inspected and, when necessary, cleaned.

Bending. 61. Reinforcement shall be carefully formed to the dimensions indicated on the plans or as provided in Section 140. Cold bends shall be made around a pin having a diameter of four or more times the least dimension of the reinforcement bars for steel of structural grade and eight or more times that for steel of intermediate or hard grade.

Straightening. 62. Metal reinforcement shall not be bent or straightened in a manner that will injure the material. Bars with kinks or bends not shown on the plans shall not be used. Heating of reinforcement will be permitted only when the entire operation is approved by the Engineer.

Placing. 63. Metal reinforcement shall be accurately positioned, and secured against displacement by using annealed iron wire of not less than No. 18 gage or suitable clips at intersections, and shall be supported by concrete or metal chairs or spacers, or metal hangers. The minimum clear distance between parallel bars shall be $1\frac{1}{2}$ times the diameter of round bars or $1\frac{1}{2}$ times the diagonal of square bars; if the ends of bars are anchored as specified in Section 140, the clear spacing may be made equal to the diameter of round bars or to the diagonal of square bars, but in no case shall the spacing between bars be less than 1 in., nor less than $1\frac{1}{4}$ times the maximum size of the coarse aggregate. Bars parallel to the face of any member shall be embedded a clear distance of not less than one diameter from the face.

Splicing. 64. In slabs, beams and girders, splices of reinforcement shall not be made at points of maximum stress without the approval of the Engineer. Splices, where permitted, shall provide sufficient lap to transfer the stress between bars by bond and shear. In such splices the bars shall be placed at the minimum distance specified in Section 63; adjacent bars shall not be spliced at the same point.

Splices in columns, piers and struts shall provide sufficient lap to transfer the stress by bond.

Offsets in Column Reinforcement. 65. Where changes in the cross-section of a compression member occur, the longitudinal reinforcement bars shall be sloped for the full

length of the member or offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion from the axis of the member shall not be more than 1 in 6.

66. Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion. **Future Bonding.**

B. Protective Concrete Covering.

67. Metal reinforcement in wall footings and column footings shall have a minimum covering of 3 in. of concrete. At surfaces of concrete exposed to the weather, metal reinforcement shall be protected by not less than 2 in. of concrete. **Moisture Protection.**

68. Metal reinforcement in fire-resistive construction shall be protected by not less than 1 in. of concrete in slabs and walls, and not less than 2 in. in beams, girders and columns, provided aggregate showing an expansion not materially greater than that of limestone or trap rock is used; when impracticable to obtain aggregate of this grade, the protective covering shall be 1 in. thicker and shall be reinforced with metal mesh having openings not exceeding 3 in., placed 1 in. from the finished surface. **Fire Protection.**

In structures where the fire hazard is limited, the metal reinforcement shall not be placed nearer the exposed surface than $\frac{3}{4}$ in. in slabs and walls or $1\frac{1}{2}$ in. in beams, girders and columns.

C. Joints.

69. Joints not indicated on the plans shall be so designed and located as to least impair the strength and appearance of the structure. To prevent laitance in horizontal joints, excess water shall be removed from the surface forming the joint after depositing the concrete. Surfaces of contact shall be cleaned and wetted before depositing is resumed, and any laitance shall be removed. Where additional resistance to horizontal shear is required, stones shall be partially embedded in such a manner as to key with the adjoining concrete; or mortices or keys shall be formed in the concrete. **General.**

70. Joints in columns shall be made at the underside of the floor. Haunches and column capitals shall be considered as part of and to act continuous with the floor. At least two hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs. **Joints in Columns.**

71. Construction joints in floors shall be located near the middle of spans of slabs, beams or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a **Joints in Floors.**

distance equal to twice the width of the beam. Adequate provision shall be made for shear by use of inclined reinforcement.

Construction Joints in Long Buildings.

72. Construction joints made crosswise of a building 100 ft. or more in length, shall have special reinforcement placed at right angles to the joint and extending a sufficient distance on each side of the joint to develop the strength of the reinforcement by bond. This reinforcement shall be placed near the opposite face of the member from the main tension reinforcement; the cross-sectional area of such reinforcement shall be not less than 0.5 per cent of the section of the members cut by the joint.

Expansion Joints.

73. Expansion joints shall be so detailed that the necessary movement may occur with the minimum resistance at the joint. The structure adjacent to the joint shall preferably be supported on separate columns or walls. Reinforcement shall not extend across an expansion joint; the break between the two sections shall be complete.[1] Exposed edges of expansion joints in walls or abutments shall be rounded. Exposed expansion joints between two distinct concrete members shall be filled with an elastic joint filler of approved quality.

Expansion Joints in Long Buildings.

74. Buildings exceeding 200 ft. in length and of width less than about one-half the length, shall be divided by means of expansion joints, located near the middle, but not more than 200 ft. apart, to minimize the destructive effects of temperature changes and shrinkage. Where there is an abrupt change in the width of a building, an expansion joint shall be provided.

Sliding Joints.

75. The seat of sliding joints shall be finished to a smooth plane surface and allowed to harden. Two thicknesses of building paper shall be placed on the seat before depositing superimposed concrete.[2]

76. Where construction joints are required to be water-tight the method of construction shall be as follows:

Water-tight Construction Joints.

(*a*) Horizontal joints shall be constructed by forming a continuous keyway in the lower portion of concrete before the concrete has hardened. Before placing the superimposed concrete the joint shall be thoroughly cleaned of laitance or other foreign material, saturated with water and coated with neat cement grout. The superimposed concrete shall be placed in such a manner as will insure an excess of mortar over the entire surface of the joint.

(*b*) Vertical joints shall be made by a metal water-stop approved by the Engineer.

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[1] A coating of white lead and oil, asphalt paint, petrolatum, or waterproofed building paper, placed over the entire surface of the hardened concrete, is commonly used for this purpose.

[2] Sheet zinc, lead, and bronze are also used for this purpose.

Seepage water shall be collected and drained from the forms; where required, vent pipes shall be closed after the concrete has thoroughly hardened.[1]

CHAPTER IX. WATERPROOFING AND PROTECTIVE TREATMENT

A. Waterproofing.

77. Concrete required to be water-tight shall be made with strict adherence to all provisions in these specifications regarding the choice of materials, proportions, consistency, mixing, placing, protecting, and workmanship. **General.**

78. Integral compounds shall not be used for waterproofing unless specifically authorized by the Engineer. **Integral Compounds.**

79. See Section 76. **Water-tight Joints.**

B. Oilproofing.

80. Concrete containers for light mineral oils, animal oils, certain vegetable oils and other commercial liquids shall be given an inside coating which shall be applied before the container is placed in service; the coating and the method of application shall be approved by the Engineer.[2] Floors or other surfaces exposed to heavy concentrations of such oils or liquids shall be similarly protected. **Oilproofing.**

C. Concrete in Sea Water.

81. Plain concrete in sea water from 2 ft. below low water to 2 ft. above high water, or from a plane below to a plane above wave action, shall contain a minimum of $1\frac{3}{4}$ bbl.(7 bags) of portland cement per **Proportions.**

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[1] To secure water-tight joints it is vitally essential that all incoming water be drained from behind the forms during the process of concreting. If the upper portion of the joint is concreted first, a metal water-stop should be provided as directed by the Engineer.

[2] The effect of certain oils and other liquids on concrete and surface treatments which have been found beneficial are given in Appendix 17.

Concrete containers for mineral oils of 30° Baumé and lighter should be given a coating which will not be affected or penetrated by mineral spirits. Coatings containing ingredients which saponify or oxidize in the presence of lime shall be applied only to a surface which has previously been treated to neutralize the lime.

Concrete containers for mineral oils heavier than 30° Baumé require no coating.

Certain vegetable and animal oils and strong acids and alkaline solutions have a destructive effect on concrete and some protective treatment is required. Several vegetable oils have a disintegrating effect on concrete in cases where the surface is alternately wet and dry, yet when stored in closed concrete containers have no injurious action.

Concrete containers for commercial liquids involve certain special features in addition to the general requirements of good concrete. Containers for such liquids often have thin sections, therefore a rich mixture is necessary to obtain concrete of proper workability without sacrificing strength and impermeability; it is especially necessary to eliminate joints or seams; thorough spading is essential and should be continuous during the placing of concrete.

cubic yard in place. Other plain concrete in sea water or exposed directly along the sea coast shall contain a minimum of $1\frac{1}{2}$ bbl. (6 bags) of portland cement per cubic yard in place. Porous or weak aggregates shall not be used.

Consistency. 82. The consistency shall meet the requirements of Section 30.

Depositing. 83. Sea water shall not be allowed to come in contact with the concrete until it has hardened for at least four days. Concrete shall be placed in such a manner as to minimize the number of horizontal or inclined seams or work planes. The placing of concrete between tides shall be a continuous operation, in accordance with the methods described in Section 43; where it is impossible to avoid seams or joints proceed as in Section 44. Concrete shall be deposited in sea water only when so directed by the Engineer, in which case it shall be placed in accordance with the methods described in Sections 47 to 52.

Protection. 84. Metal reinforcement shall be placed at least 3 in. from any plane or curved surface, except at corners when it shall be at least 4 in. from adjacent surfaces. Metal chairs, supports, or ties shall not extend to the surface of the concrete. Where unusually severe conditions of abrasion are anticipated, the face of the concrete from 2 ft. below low water to 2 ft. above high water, or from a plane below to a plane above wave action, shall be protected by creosoted timber, dense vitrified shale brick, or stone of suitable quality, as designated on the plans or as required by the Engineer.

D. Concrete in Alkali Soils or Waters.[1]

Proportions. 85. Concrete in alkali waters or below ground-line of alkali soils shall contain a minimum of $1\frac{3}{4}$ bbl. (7 bags) of portland cement per cubic yard in place.[2]

Consistency. 86. The consistency of concrete in alkali soils or waters shall be such as to meet the requirements of Section 30.

Placing. 87. Concrete shall be placed in such a manner as to minimize the number of horizontal or inclined seams, or work planes.

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[1] Under certain circumstances concrete is attacked by alkaline waters. The term "alkali" is here used to designate the soluble salts which occur in considerable quantities in the soils and waters of certain Western States; the sulfates, chlorides, and carbonates of sodium and magnesium are the most common forms of alkali.

It is important to distinguish between the different forms of alkali which occur in nature. Experience and tests have shown that certain forms are more injurious than others; the chlorides and carbonates produce little or no injury to concrete.

Special care must be used in placing concrete where it will be exposed to sulfate waters. An impermeable concrete made with a durable aggregate is necessary. Concrete should be permitted so harden under favorable conditions before it is exposed to injurious alkalies, and wherever practicable such concrete should be made in the form of precast units.

[2] Where the foundations of important buildings or similar structures are subject to high concentrations of alkalies, under-drainage may be used as an added precaution.

88. Metal reinforcement or other corrodible metal shall not be placed closer than 2 in. to the surface of members exposed to alkali soils or waters. In foundations and in heavy structures the metal reinforcement shall not be placed closer than 3 in. to the surface. **Protection.**

CHAPTER X. SURFACE FINISH[1]

89. The requirements in these specifications applying to forms, and to mixing, conveying, depositing and finishing concrete, shall be followed unless modified by the plans. **General.**

The whole of a showing face between prescribed construction joints shall be cast in one continuous operation. Construction joints, when not shown on the plans, shall be made as directed by the Engineer, and shall be true to line with sharp unbroken edges.

The same brand of cement, and the same kind and size of aggregate, shall be used throughout the whole of any showing face.

For showing faces the forms shall be smooth and watertight. If wood be used, the boards shall be planed, grooved and tongued, evenly matched and tightly placed. They shall be so constructed as to be removable in sections by unscrewing or otherwise loosening them without hammering or prying against the face. Any offsets in the forms that may occur shall be smoothly dressed and any openings pointed flush with stiff clay or plaster of Paris in order to prevent leakage or the formation of fins.

Concrete that is to have a showing face, whether any particular finish is called for or not, shall be mixed, placed and worked as may be necessary to secure at the face a uniform distribution of the aggregates, freedom from void spaces, and uniform texture. If the finish is required to be one that will expose the coarse aggregate, by either scrubbing, tooling, sand-blasting, or acid treatment, then after the full surface of mortar has been worked against the form, the spading tool shall be inserted in the concrete and the coarse aggregate be pressed against the form in order to secure uniform distribution at the face and a uniform texture after the aggregate is exposed.

Face forms shall be removed as soon as practicable in order to facilitate effective repair of void spaces or broken corners, before the surface has dried. Care shall be taken to avoid roughening or injuring corners, and to keep edges sharp.

As soon as the face forms are removed any fins or other projections shall be carefully removed, and offsets leveled, and any voids or

[1] The Engineer must indicate on the plans the type of finish desired and must strike out the sections which do not apply.

damaged places shall immediately be saturated with water and filled with a mixture of the same composition as that used in the surface, and brought even with the surface by means of a wooden spatula or float. A steel trowel shall not be used to finish the surface. The face shall be finished free from streaks, discolorations or other imperfections. Plastering will not be permitted.

Where a surface of mortar is to be the basis of the finish the coarse aggregate shall be worked back from the form with a suitable tool, so as to bring a full surface of mortar against the form, care being taken to prevent the formation of voids and air pockets.

Whenever forms are removed from showing faces before the concrete has become hard and dry, the surface of the concrete shall be immediately wetted and kept wet for at least three days.

Granolithic Surfaces.

90. Granolithic surfaces shall be made by placing about 1 in. of facing concrete against the face form in advance of the concrete, of such consistency and in such a manner as will insure its bonding with the concrete.

The facing concrete shall be composed of 1 part portland cement, $1\frac{1}{2}$ parts fine aggregate, and $2\frac{1}{2}$ parts coarse aggregate made up of pebbles, crushed granite, or other stone as called for.

If iron or wooden molds are used to retain the facing against the forms while placing concrete, care should be taken that the mold is not permitted to remain until initial setting occurs. The molds shall be jarred frequently and raised at short intervals to prevent formation of seams and air spaces between the surface and the concrete.

Top Surfaces not Subject to Wear.

91. Top surfaces not subject to wear shall be smoothed with a wood float and be kept wet for at least seven days. Care shall be taken to avoid an excess of water in the concrete, and to drain or otherwise promptly remove any water that comes to the surface. Dry cement, or a dry mixture of cement and sand, shall not be sprinkled directly on the surface.

A. *Wearing Surfaces.*

One-Course Work.

92. Aggregates for the wearing surface shall have a high resistance to abrasion, and shall be screened and when necessary thoroughly washed. The least quantity of mixing water that will produce a dense concrete shall be used. The mix shall not be leaner than 1 part of Portland cement and $2\frac{1}{2}$ parts of aggregate. The surface shall be screeded even and finished with a wood float. Excess water shall be promptly drained or otherwise removed. Overtroweling shall be avoided.

93. The wearing surface in two-course work shall be placed within ½ hour after the base course. Where the wearing surface is required to be applied to a hardened base course, the latter shall be prepared by roughening with a pick or other effective tool. The roughened surface shall be thoroughly saturated with water and covered with a thin layer of neat cement paste immediately before the wearing surface is placed. The wearing course shall not be thinner than 1 in. **Two-Course Work.**

94. Concrete wearing surfaces made in accordance with Sections 92 and 93, shall be kept wet[1] for at least 10 days in the case of floors and 21 days in the case of roads and pavements. **Curing.**

95.[2] Terrazzo finish shall be made by mixing 1 part of cement, 2½ parts of crushed marble, or other stone or crushed pebbles, as may be called for by the plans, and sufficient water to produce a dense concrete. The concrete shall be spread on the base course and worked down to a thickness of 1 in. by patting or rolling and troweling. The marble shall all pass a ½-in. screen and be free from dust. The surface shall be kept wet for not less than ten days, and after curing shall be rubbed to a plane surface with a stone or a surfacing machine. Hardened concrete to which a terrazzo finish is to be applied shall be prepared as prescribed in Section 93. **Terrazzo Finish.**

95.[2] Terrazzo finish shall be made by mixing 1 part of cement, 2 parts of sand and sufficient water to produce a plastic mortar, which shall be spread on the base course to a depth of 1 in. Crushed marble, free from dust and passing a ¼-in. screen, shall be sprinkled over the surface of the fresh mortar and pressed or rolled in. The surface shall be kept wet for not less than ten days and after curing shall be rubbed to a plane surface with a stone or a surfacing machine. Hardened concrete to which a terrazzo finish is to be applied shall be prepared as prescribed in Section 93. **Terrazzo Finish.**

B. Decorative Finishes.

96. Immediately after the forms are removed, the surface shall be wetted and rubbed with a carborundum brick or other abrasive until even and smooth and of uniform appearance, without applying any cement or other coating. **Rubbed Finish.**

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[1] Prevention of premature drying during the early hardening of concrete is essential to the development of high resistance to abrasion. The surface may be covered with a layer of burlap, earth or sand, kept wet, or it may be divided into small areas by dikes and flooded with water to a depth of 2 or 3 in.

[2] The Engineer must strike out one of the Sections 95.

Scrubbed Finish. 97. The forms shall be removed and the scrubbing done before the concrete has hardened.[1] The surface shall be scrubbed with fiber or wire brushes using water freely, until the surface film of mortar is removed and the aggregate uniformly exposed; then rinsed with clean water. If portions of the surface have become too hard to scrub in equal relief, dilute hydrochloric acid (1 part acid to 4 parts water) may be used to facilitate the scrubbing. The remaining acid shall be thoroughly removed with clean water.

Sand-Blast Finish. 98. The concrete face shall be permitted to attain an intermediate degree of hardness; it shall then be air-blasted with hard sand until the aggregate is in uniform relief.

Tooled Finish. 99. The surface shall be permitted to become dry and hard, and then dressed with tools, as called for,[2] to a uniform texture and even face.

Sand Floated Finish. 100. The forms shall be removed before the surface has fully hardened; the surface shall be rubbed with a wooden float by a uniform circular motion, fine sand being rubbed into the surface until the resulting finish is even and uniform.

Colored Aggregate Finish. 101. Colored or other special aggregate used for finish shall be exposed by scrubbing as provided in Section 97. Facing mortar, made from this special aggregate, of 1 part of cement, $1\frac{1}{2}$ parts of sand, and 3 parts of pebbles or screenings, clean and free from dust, shall be placed against the face forms to a thickness of about 1 in., sufficiently in advance of the concrete to prevent the latter coming in contact with the form.

Colored Pigment Finish. 102. Mineral pigment shall be thoroughly mixed dry with the cement and fine aggregate; care shall be taken to secure a uniform tint throughout.

CHAPTER XI. DESIGN.

A. *General Assumptions.*

General Assumptions. 103. The design of reinforced concrete members under these specifications shall be based on the following assumptions:

(*a*) Calculations are made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.

(*b*) A plane section before bending remains plane after bending, shearing distortions being neglected.

(*c*) The modulus of elasticity of concrete in compression is con-

[1] In warm weather this will require from 6 to 24 hours, and in cold weather from 1 to 3 days.

[2] By axing, rough or fine pointing, or bush-hammering.

stant within the limits of working stresses and the distribution of compressive stress in beams is rectilinear.

(*d*) The moduli of elasticity of concrete in computations for the position of the neutral axis, for the resisting moment of beams, and for compression of concrete in columns, are as follows:

(1) $\frac{1}{15}$ that of steel, when the compressive strength of the concrete at 28 days exceeds 1500 and does not exceed 2200 lb. per sq. in.;

(2) $\frac{1}{12}$ that of steel, when the compressive strength of the concrete at 28 days exceeds 2200 and does not exceed 2900 lb. per sq. in.;

(3) $\frac{1}{10}$ that of steel, when the compressive strength of the concrete at 28 days is greater than 2900 lb. per sq. in.

(*e*) In calculating the moment of resistance of reinforced concrete beams and slabs the tensile resistance of the concrete is neglected.

(*f*) The bond between the concrete and the metal reinforcement remains unbroken throughout the range of working stresses. Under compression the two materials are therefore stressed in proportion of their moduli of elasticity.

(*g*) Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected, except in the design of reinforced concrete columns.[1]

B. *Flexure of Rectangular Reinforced Concrete Beams and Slabs.*

104. Computations of flexure in rectangular reinforced concrete beams and slabs shall be based on the following formulas: **Flexure Formulas.**

(*a*) *Reinforced for Tension Only.*

Position of neutral axis,

$$k=\sqrt{2pn+(pn)^2}-pn \quad \ldots\ldots (1)$$

Arm[2] of resisting couple,

$$j=1-\frac{k}{3} \quad \ldots\ldots (2)$$

Compressive unit stress[2] in extreme fiber of concrete,

$$f_c=\frac{2M}{jkbd^2}=\frac{2pf_s}{k} \quad \ldots\ldots (3)$$

[1] Formula 43 for the permissible compressive stress in reinforced concrete columns takes into account the effect of shrinkage in the concrete on the stress in the longitudinal reinforcement. It is not required, however, that the designer consider shrinkage stresses in columns, except through the use of that formula.

[2] For f_s = 16,000 to 18,000 lb. per sq. in. and f_c = 800 to 900 lb. per sq. in., j may be assumed as 0.86. For values of pn varying from 0.04 to 0.24, jk is approximately equal to $0.67\sqrt[3]{pn}$.

Tensile unit stress[1] in longitudinal reinforcement,

$$f_s = \frac{M}{A_s jd} = \frac{M}{pjbd^2} \quad \text{.....................(4)}$$

Steel ratio for balanced reinforcement,

$$p = \frac{1}{2} \frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)} \quad \text{.....................(5)}$$

For formulas on shear and bond, see Sections 121–127 and 135–137.

(*b*) *Reinforced for Both Tension and Compression.*

Position of neutral axis,

$$k = \sqrt{2n\left(p + p'\frac{d'}{d}\right) + n^2(p + p')^2} - n(p + p') \quad \text{.........(6)}$$

Position of resultant compression,

$$z = \frac{\frac{1}{3}k^3 d + 2p'nd'\left(k - \frac{d'}{d}\right)}{k^2 + 2p'n\left(k - \frac{d'}{d}\right)} \quad \text{.................(7)}$$

Arm[1] of resisting couple,

$$jd = d - z \quad \text{........................(8)}$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{6M}{bd^2\left[3k - k^2 + \frac{6p'n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]} \quad \text{...........(9)}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{pjbd^2} = nf_c\frac{1-k}{k} \quad \text{.................(10)}$$

Compressive unit stress in longitudinal reinforcement,

$$f'_s = nf_c \frac{k - \frac{d'}{d}}{k} \quad \text{.....................(11)}$$

[1] For f_s=16,000 to 18,000 lb. per sq. in. and f_c=800 to 900 lb. per sq. in., j may be assumed as 0.86. For values of pn varying from 0.04 to 0.24, jk is approximately equal to $0.67\sqrt[3]{pn}$.

105. The symbols[1] used in Formulas 1 to 23 are defined as follows: **Notation.**

A_s = effective cross-sectional area of metal reinforcement in tension in beams;

b = width of rectangular beam or width of flange of T-beam;

d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement;

d' = depth from compression surface of beam or slab to center of compression reinforcement;

f_c = compressive unit stress in extreme fiber of concrete;

f_s = tensile unit stress in longitudinal reinforcement;

f'_s = compressive unit stress in longitudinal reinforcement;

h = unsupported length of column;

I = moment of inertia of a section about the neutral axis for bending;

j = ratio of lever arm of resisting couple to depth d;

k = ratio of depth of neutral axis to depth d;

l = span length of beam or slab (generally distance from center to center of supports—see Section 106);

M = bending moment or moment of resistance in general;

n = E_s/E_c = ratio of modulus of elasticity of steel to that of concrete;

p = ratio of effective area of tension reinforcement to effective area of concrete in beams = A_s/bd;

p' = ratio of effective area of compression reinforcement to effective area of concrete in beams;

w = uniformly distributed load per unit of length of beam or slab;

z = depth from compression surface of beam or slab of resultant of compressive stresses.

106. The span length, l, of freely supported beams and slabs, **Span Length.** shall be the distance between centers of the supports, but shall not exceed the clear span plus the depth of beam or slab. The span length for continuous or restrained beams built to act integrally with supports shall be the clear distance between faces of supports. Where brackets having a width not less than the width of the beam and making an angle of 45 deg. or more with the horizontal axis of a restrained beam are built to act integrally with the beam and support, the span shall be measured from the section where the combined depth of the beam and bracket is at least one-third more than the depth of the beam, but no portion of such a bracket shall be considered as adding

[1]For illustration of notation as applied to typical beams or slabs, see Figs. 1 and 2.

to the effective depth of the beam. Maximum negative moments are to be considered as existing at the ends of the span, as defined above.

Slightly Restrained Beams of Equal Span.

107. Beams and slabs of equal spans built to act integrally with beams, girders, or other slightly restraining supports and carrying uniformly distributed loads shall be designed for the following moments at critical sections:

(*a*) Beams and slabs of one span,
Maximum positive moment near center,

$$M = \frac{wl^2}{8} \quad \text{(12)}$$

(*b*) Beams and slabs continuous for two spans only,
(1) Maximum positive moment near center,

$$M = \frac{wl^2}{10} \quad \text{(13)}$$

(2) Negative moment over interior support,

$$M = \frac{wl^2}{8} \quad \text{(14)}$$

(*c*) Beams and slabs continuous for more than two spans,
(1) Maximum positive moment near center and negative moment at support of interior spans,

$$M = \frac{wl^2}{12} \quad \text{(15)}$$

(2) Maximum positive moment near centers of end spans and negative moment at first interior support,

$$M = \frac{wl^2}{10} \quad \text{(16)}$$

(*d*) Negative moment at end supports for cases (*a*), (*b*), (*c*) of this section,

$$M = \text{not less than } \frac{wl^2}{16} \quad \text{(16}a\text{)}$$

Beams Built Into Brick or Masonry Walls.

108. Beams and slabs built into brick or masonry walls in a manner which develops partial end restraint shall be designed for a negative moment at the support of

$$M = \text{not less than } \frac{wl^2}{16} \quad \text{(17)}$$

109. Beams and slabs of equal spans freely supported and assumed to carry uniformly distributed loads shall be designed for the moments specified in Section 107, except that no reinforcement for negative moment need be provided at end supports where effective measures are taken to prevent end restraint. The span shall be taken as defined in Section 106 for freely supported beams. **Freely Supported Beams of Equal Span.**

110. Beams and slabs of equal span built to act integrally with columns, walls, or other restraining supports and assumed to carry uniformly distributed loads, shall (except as provided in Section 107) be designed for the following moments at critical sections: **Restrained Beams of Equal Span.**

(*a*) Interior spans,

(1) Negative moment at interior supports except the first,

$$M = \frac{wl^2}{12} \qquad (18)$$

(2) Maximum positive moment near centers of interior spans,

$$M = \frac{wl^2}{16} \qquad (19)$$

(*b*) End spans of continuous beams and beams of one span in which I/l is less than twice the sum of the values of I/h for the exterior columns above and below which are built into the beams:

(1) Maximum positive moment near center of span and negative moment at first interior supports,

$$M = \frac{wl^2}{12} \qquad (20)$$

(2) Negative moment at exterior supports,

$$M = \frac{wl^2}{12} \qquad (21)$$

(*c*) End spans of continuous beams, and beams of one span, in which I/l is equal to or greater than twice the sum of the values of I/h for the exterior column above and below which are built into the beams:

(1) Maximum positive moment near center of span and negative moment at first interior support,

$$M = \frac{wl^2}{10} \qquad (22)$$

(2) Negative moment at exterior support,

$$M = \frac{wl^2}{16} \quad \text{.......................} (23)$$

Continuous Beams of Unequal Spans or with Non-uniform Loading.

111. Continuous beams with unequal spans, or with other than uniformly distributed loading, whether freely-supported or restrained, shall be designed for the actual moments under the conditions of loading and restraint.

Provision shall be made where necessary for negative moment near the center of short spans which are adjacent to long spans, and for the negative moment at the end supports, if restrained.

Unsupported Flange Length.

112. The distance between lateral supports of the compression area of a beam shall not exceed 24 times the least width of compression flange.

C. *Flexure of Reinforced Concrete T-Beams.*

Flexure Formulas.

113. Computations of flexure in reinforced concrete T-beams shall be based on the following formulas:

(*a*) *Neutral Axis in the Flange.*

Use formulas for rectangular beams and slabs in Section 104.

(*b*) *Neutral Axis below the Flange.*

Position of neutral axis,

$$kd = \frac{2ndA_s + bt^2}{2nA_s + 2bt} \quad \text{.......................} (24)$$

Position of resultant compression,

$$z = \left(\frac{3kd - 2t}{2kd - t}\right)\frac{t}{3} \quad \text{....................} (25)$$

Arm of resisting couple,

$$jd = d - z \quad \text{.........................} (26)$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{Mkd}{bt(kd - \frac{1}{2}t)jd} = \frac{f_s}{n}\left(\frac{k}{1-k}\right) \quad \text{..............} (27)$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s jd} \quad \text{.......................} (28)$$

Formulas 24, 25, 26, 27 and 28 neglect compression in the stem.[1]

Notation. 114. The symbols[2] used in Formulas 24 to 28 are defined in Section 105, except as follows:

b' = width of stem of T-beam;
t = thickness of flange of T-beam.

Flange Width. 115. Effective and adequate bond and shear resistance shall be provided in beam-and-slab construction at the junction of the beam and slab; the slab shall be built and considered an integral part of the beam; the effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth of the span length of the beam, and its overhanging width on either side of the web shall not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

For beams having a flange on one side only, the effective flange width to be used in design shall not exceed one-tenth of the span length of the beam, and its overhanging width from the face of the web shall not exceed six times the thickness of the slab nor one-half the clear distance to the next beam.

Transverse Reinforcement. 116. Where the principal slab reinforcement is parallel to the beam, transverse reinforcement, not less in amount than 0.3 per cent of the sectional area of the slab, shall be provided in the top of the slab and shall extend across the beam and into the slab not less than two-thirds of the width of the effective flange overhang. The spacing of the bars shall not exceed 18 in.

Compressive Stress at Supports. 117. Provision shall be made for the compressive stress at the support in continuous T-beam construction.

Shear. 118. The flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

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[1] The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$kd=\sqrt{\frac{2ndA_s+(b-b')t^2}{b'}+\left(\frac{nA_s+(b-b')t}{b'}\right)^2}-\frac{nA_s+(b-b')t}{b'} \qquad \text{(24a)}$$

Position of resultant compression,

$$z=\frac{(kdt^2-\frac{2}{3}t^3)b+[(kd-t)^2(t+\frac{1}{3}(kd-t))]\,b'}{t(2kd-t)b+(kd-t)^2b'} \qquad \text{(25a)}$$

Arm of resisting couple (see footnote Section 106),

$$jd=d-z \qquad \text{(26a)}$$

Compressive unit stress in extreme fiber of concrete,

$$f_c=\frac{2Mkd}{[(2kd-t)bt+(kd-t)^2b']\,jd} \qquad \text{(27a)}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s=\frac{M}{A_s jd} \qquad \text{(28a)}$$

[2] For illustration of certain symbols as applied to typical T-beams, see Fig. 3, Appendix 1.

Isolated Beams. 119. Isolated beams in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web and a total flange width not more than 4 times the web thickness.

D. *Diagonal Tension and Shear.*

Notation. 120. The symbols used in Formulas 29 to 35 are defined in Section 105, except as follows:

A_v = total area of web reinforcement in tension within a distance s, that is $s_1, s_2, s_3, \ldots s_n$, or the total area of all bars bent up in any one plane;
α = angle between web bars and longitudinal bars;
F = total tension in a bar;
f'_c = ultimate compressive strength of concrete at age of 28 days, based on tests of 6 by 12-in. or 8 by 16-in. cylinders made and tested in accordance with the Standard Methods of Making and Storing Specimens of Concrete in the Field (Appendix 13) and the Tentative Methods of Making Compression Tests of Concrete (Appendix 12);
f_v = tensile unit stress in web reinforcement;
Q = ratio of the average to the maximum bond stress computed by Formula 34 within the distance y;
Σo = sum of perimeters of bars in one set;
r = ratio of cross-sectional area of negative reinforcement which crosses entirely over the column capital of a flat slab or over the dropped panel, to the total cross-sectional area of the negative reinforcement in the two column strips;
s = spacing of web bars or stirrups measured at the plane of the lower reinforcement and in the direction of the longitudinal axis of the beam;
t_1 = thickness of flat slab without dropped panels or thickness of a dropped panel (see Appendix 1, Fig. 14);
t_2 = thickness of flat slab with dropped panels at points away from the dropped panel (see Appendix 1, Fig. 14);
u = bond stress per unit of area of surface of bar;
v = shearing unit stress;
V = total shear;
x = length of bar added for anchorage, including the hook, if any;
y = distance from the point at which tension is computed to the point of beginning of anchorage.

121. The shearing unit-stress, v, in reinforced concrete beams shall be taken as not less than that computed by Formula 29.[1] **Formula for Shear.**

$$v = \frac{V}{bjd} \quad \text{. . . . . (29)}$$

122. For purpose of design of beams carrying uniform loads, not less than one-fourth of the total shearing resistance required at either end of span shall be provided at the section where the computed shearing stress is zero; from that section to the ends of span the required shearing resistance shall be assumed to vary uniformly. **Variation of Shear in Beams with Uniform Load.**

123. The shearing unit stress shall be computed on the minimum width of rectangular beams and on the minimum thickness of the web in beams of I or T-section. **Width of Beams in Shear Computations.**

124. The width of the effective section for shear as governing diagonal tension shall be assumed as the thickness of the concrete web plus one-half the thickness of the vertical webs of the concrete or clay tile in contact with the beam. (For typical design see Appendix 1, Fig. 13.) **Shear in Beam-and-Tile Construction.**

125. Web reinforcement may consist of: **Types and Spacing of Web Reinforcement.**

(*a*) Vertical stirrups or web reinforcing bars;

(*b*) Inclined stirrups or web reinforcing bars forming an angle of 30 deg. or more with the longitudinal bars;

(*c*) Longitudinal bars bent up at an angle of 15 deg. or more with the direction of the longitudinal bars.

Stirrups or bent-up bars which are not anchored at both ends, according to the provisions of Section 141, shall not be considered effective as web reinforcement. When the shearing stress is not greater than 0.06 f'_c, the distance s measured in the direction of the axis of the beam between two successive stirrups, or between two successive points of bending up of bars, or from the point of bending up of a bar to the edge of the support, shall not be greater than

$$s = \frac{45d}{\alpha + 10} \quad \text{. (30)}$$

where the angle α is in degrees.

When the shearing stress is greater than $0.06f'_c$, the distance s shall not be greater than two-thirds of the values given by Formula 30.

126. See Section 141. **Anchorage of Web Reinforcement.**

[1] For I or T-beams b is the width of the stem as given in Section 123.

Beams Without Special Anchorage of Longitudinal Reinforcement.

127. The shearing unit stress computed by Formula 29 in beams in which the longitudinal reinforcement is without special anchorage shall not exceed the values given by Formulas 31 and 32 and in no case shall it exceed $0.06f'_c$.

When α is between 45 and 90 deg.,

$$v = 0.02f'_c + \frac{f_v A_v}{bs \sin \alpha} \qquad (31)$$

When α is less than 45 deg.,

$$v = 0.02f'_c + \frac{f_v A_v}{bs} (\sin \alpha + \cos \alpha) \qquad (32)$$

Beams With Special Anchorage of Longitudinal Reinforcement.

128. The shearing unit stress computed by Formula 29 in beams in which longitudinal reinforcement is anchored by means of hooked ends or otherwise, as specified in Section 140, shall not exceed the value given by Formulas 31 and 32, when $0.03f'_c$ is substituted for $0.02f'_c$ in those formulas; in no case shall the shearing unit stress exceed $0.12f'_c$.

Beams with Bars Bent up at a Single Point.

129. Where the web reinforcement consists of bars bent up at a single point, the point of bending shall be at a distance s from the edge of the support, not greater than that given in Section 125 and the value of the quantity $\frac{f_v A_v}{bs} (\sin \alpha + \cos \alpha)$ used in the design shall not exceed 75 lb. per sq. in. (See Appendix 1, Fig. 10.)

Combined Web Reinforcement.

130. Where two or more types of web reinforcement are used in conjunction, the total shearing resistance of the beam shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete (the term $0.02f'_c$ or $0.03f'_c$ in Formulas 31 and 32) shall be included only once. In no case shall the maximum shearing stresses be greater than the limiting values given in Sections 127 and 128.

Shearing Stress in Flat Slabs.

131. The shearing unit stress in flat slabs shall not exceed the value of v as given by Formula 33,

$$v = 0.02\, f'_c\,(1+r) \qquad (33)$$

and shall not in any case exceed $0.03f'_c$.

The shearing unit stress shall be computed on:

(*a*) A vertical section which has a depth in inches of $\frac{7}{8}\,(t_1 - 1\frac{1}{2})$ and which lies at a distance in inches of $t_1 - 1\frac{1}{2}$ from the edge of the column capital; and

(*b*) A vertical section which has a depth in inches of $\frac{7}{8}\,(t_2 - 1\frac{1}{2})$ and which lies at a distance in inches of $t_2 - 1\frac{1}{2}$ from the edge of the dropped panel.

In no case shall r be less than 0.25. Where the shearing stress computed as in (a) is being considered, r shall be assumed as the proportional amount of the negative reinforcement, within the column strip, crossing the column capital. Where the shearing stress computed as in (b) is being considered, r shall be assumed as the proportional amount of the negative reinforcement, within the column strip, crossing entirely over the dropped panel.[1] (For typical flat slab and designation of principal design sections see Appendix 1, Figs. 14 and 15.)

Shear and Diagonal Tension in Footings.

132. The shearing stress shall be taken as not less than that computed by Formula 29. The stress on the critical section, shall not exceed $0.02f'_c$ for footings with straight reinforcement bars, nor $0.03f'_c$ for footings in which the reinforcement bars are anchored at both ends by adequate hooks or otherwise as specified in Section 140.

Critical Section for Soil Footings.

133. The critical section for diagonal tension in footings on soil shall be computed on a vertical section through the perimeter of the lower base of a frustum of a cone or pyramid which has a base angle of 45 deg., and which has for its top the base of the column or pedestal and for its lower base the plane at the centroid of longitudinal reinforcement.

Critical Section for Pile Footings.

134. The critical section for diagonal tension in footings on piles shall be computed on a vertical section at the inner edge of the first row of piles entirely outside a section midway between the face of the column or pedestal and the section described in Section 133 for soil footings, but in no case outside of the section described in Section 133. The critical section for piles not arranged in rows shall be taken midway between the face of the column and the perimeter of the base of the frustum described in Section 133.

E. Bond and Anchorage.

Bond Stresses by Beam Action.

135. Where bar reinforcement is used to resist tensile stresses developed by beam action, the bond stress shall be taken as not less than that computed by Formula 34,

$$u = \frac{V}{\Sigma ojd} \quad \text{.......................(34)}$$

For continuous or restrained members, the critical section for bond for the positive reinforcement shall be assumed to be at the point of inflection; that for the negative reinforcement shall be assumed to be at the face of the support, and at the point of inflection. For

[1] In special cases, where supported by satisfactory engineering analysis, diagonal tension reinforcement may be used and increased shearing stresses allowed in accordance with Sections 127 to 130.

simple beams or freely supported end spans of continuous beams, the critical section for bond shall be assumed to be at the face of the support.

Bent-up longitudinal bars which, at the critical section, are within a distance $\frac{d}{3}$ from horizontal reinforcement under consideration, may be included with the straight bars in computing Σo.

In footings only the bars specified in Section 177 as effective in resisting bending moment shall be considered as resisting bond stresses. Special investigation shall be made of bond stresses in footings with stepped or sloping upper surface, as maximum bond stresses may occur at the vertical plane of the steps or near the edges of the footing.

Bond Stress for Ordinary Anchorage.

136. In beams where the ordinary anchorage described in Section 139 is provided, the bond stress computed by Formula 34 at any section shall not exceed the following values:

For plain bars . . . . $u = 0.04f'_c$
For deformed bars meeting the requirements of Section 23 . . . . $u = 0.05f'_c$

Bond Stresses for Special Anchorage.

137. In beams where special anchorage of the bars is provided as specified in Section 140, bond stresses exceeding those specified in Section 136 may be used, provided the total tensile stress at a point of abrupt change in stress or at the point of maximum stress, does not exceed the value of F given by Formula 35,

$$F = Qu\Sigma oy + u\Sigma ox \qquad (35)$$

where F = total tension in the bar;
Σo = the perimeter of the bar under consideration;
Q = ratio of the average to the maximum bond stress computed by Formula 34 within the distance y.
u = permissible bond stress = $0.04f'_c$ for plain and $0.05f'_c$ for deformed bars meeting the requirements of Section 23;
x = the length of bar added for anchorage, including the hook, if any;
y = distance from the point at which the tension is computed to the point of beginning of anchorage.

The length of bar added for anchorage may be either straight or bent. The radius of bend shall not be less than four bar diameters.

138. The permissible bond stress for footings and similar members in which reinforcement is placed in more than one direction shall not exceed 75 per cent of the values in Sections 136 and 137. **Bond Stress for Reinforcement in Two or More Directions.**

139. In continuous, restrained or cantilever beams, anchorage of the tensile negative reinforcement beyond the face of the support shall provide for the full maximum tension with bond stresses not greater than those specified in Section 136. Such anchorage shall provide a length of bar not less than the depth of the beam. In the case of end supports which have a width less than three-fourths of the depth of the beam, the bars shall be bent down toward the support a distance not less than the effective depth of the beam. The portion of the bar so bent down shall be as near to the end of the beam as protective covering permits. (See Fig. 9.) In continuous or restrained beams, negative reinforcement shall be carried to or beyond the point of inflection. Not less than one-fourth of the area of the positive reinforcement shall extend into the support to provide an embedment of ten or more bar diameters. **Ordinary Anchorage Requirements.**

In simple beams or freely supported end spans of continuous beams at least one-fourth of the area of the tensile reinforcement shall extend along the tension side of the beam and beyond the face of the support to provide an embedment of ten or more bar diameters.

140. Where increased shearing stresses are used as provided in Sections 128 and 132 or increased bond stresses as provided in Section 137, special anchorage of all reinforcement in addition to that required in Section 139 shall be provided as follows: **Special Anchorage Requirements.**

(*a*) In continuous and restrained beams, anchorage beyond points of inflection of one-third the area of the negative reinforcement and beyond the face of the support of one-third the area of the positive reinforcement, shall be provided to develop one-third of the maximum working stress in tension, with bond stresses not greater than those specified in Section 136.

(*b*) At the edges of footings, anchorage for all the bars for one-third the maximum working stress in tension shall be provided within a region where the tension in the concrete, computed as an unreinforced beam, does not exceed 40 lb. per sq. in.

(*c*) In simple beams or freely supported end spans of continuous beams, at least one-half of the tensile reinforcement shall extend along the tension side of the beam to provide an anchorage beyond the face of the support for one-third of the maximum working stress in tension.

141. Web bars shall be anchored at both ends by: **Anchorage of Web Reinforcement.**

(*a*) providing continuity with the longitudinal reinforcement; or

(*b*) bending around the longitudinal bar; or

(*c*) a semi-circular hook which has a radius not less than four times the diameter of the web bar.

Stirrup anchorage shall be so provided in the compression and tension regions of a beam as to permit the development of safe working tensile stress in the stirrup at a point $0.3d$ from either face.[1]

The end anchorage of a web member not in bearing on the longitudinal reinforcement shall be such as to engage an amount of concrete sufficient to prevent the bar from pulling out. In all cases the stirrups shall be carried as close to the upper and lower surfaces as fireproofing requirements permit. (For typical designs see Appendix 1, Figs. 8 and 12.)

F. Flat Slabs.

(*Two-Way and Four-Way Systems with Rectangular Panels.*)

Moments in Interior Panels.

142. The moment coefficients, moment distribution and slab thicknesses specified herein are for slabs which have three or more rows of panels in each direction, and in which the panels are approximately uniform in size. Slabs with paneled ceiling or with depressed paneling in the floor shall be considered as coming under the requirements herein given. The symbols used in Formulas 36 to 41 are defined in Section 105 except as indicated in Sections 142, 145 and 155.

In flat slabs in which the ratio of reinforcement for negative moment in the column strip is not greater than 0.01, the numerical sum of the positive and negative moments in the direction of either side of the panel for which tension reinforcement must be provided, shall be assumed as not less than that given by Formula 36,

$$M_0 = 0.09\ Wl \left(1 - \frac{2c}{3l}\right)^2 \qquad (36)$$

where M_0 = sum of positive and negative bending moments[2] in either rectangular direction at the principal design sections of a panel of a flat slab;

c = base diameter of the largest right circular cone, which lies entirely within the column (including the capital) whose vertex angle is 90 deg. and whose base is $1\frac{1}{2}$ in. below the bottom of the slab or the bottom of the dropped panel (see Fig. 14);

[1] Generally a properly-anchored stirrup whose diameter does not exceed $\frac{1}{50}$ of the depth of the beam will meet these requirements.

[2] The sum of the positive and negative moments provided for by this equation is about 72 per cent of the moment found by rigid analysis based upon the principles of mechanics. Extensive tests and experience with existing structures have shown that the requirements here stated will give adequate strength. See "Statical Limitations upon the Steel Requirement in Reinforced Concrete Flat Slab Floors," by John R. Nichols, *Transactions*, Am. Soc. Civil Engrs., Vol. LXXVII (1914), and "Moments and Stresses in Slabs," by Westergaard and Slater, *Proceedings*, Am. Concrete Inst., Vol. XVII (1921).

l = span length[1] of flat slab, center to center of columns in the rectangular direction in which moments are considered;

l_1 = span length[1] of flat slab, center to center of columns perpendicular to the rectangular direction in which moments are considered; and

W = total dead and live load uniformly distributed over a single panel area.

TABLE VI.—MOMENTS TO BE USED IN DESIGN OF FLAT SLABS.[2]

Strip.	Flat Slabs without Dropped Panels.		Flat Slabs with Dropped Panels.	
	Negative.	Positive.	Negative.	Positive.
SLABS WITH 2-WAY REINFORCEMENT.				
Column strip	$0.23\ M_0$	$0.11\ M_0$	$0.25\ M_0$	$0.10\ M_0$
2 Column strips	$0.46\ M_0$	$0.22\ M_0$	$0.50\ M_0$	$0.20\ M_0$
Middle strip	$0.16\ M_0$	$0.16\ M_0$	$0.15\ M_0$	$0.15\ M_0$
SLABS WITH 4-WAY REINFORCEMENT.				
Column strip	$0.25\ M_0$	$0.10\ M_0$	$0.27\ M_0$	$0.095\ M_0$
2 Column strips	$0.50\ M_0$	$0.20\ M_0$	$0.54\ M_0$	$0.190\ M_0$
Middle strip	$0.10\ M_0$	$0.20\ M_0$	$0.08\ M_0$	$0.190\ M_0$

Principal Design Sections.

143. In computing the critical moments in flat slabs subjected to uniform load the following principal design sections shall be used:

(*a*) *Section for Negative Moment in Middle Strip:* The section beginning at a point on the edge of the panel $l_1/4$ from the column center and extending in a rectangular direction a distance $l_1/2$ toward the center of the adjacent column on the same panel edge. (See Fig. 15, Appendix 1.)

(*b*) *Section for Negative Moment in Column Strip:* The section beginning at a point on the edge of the panel $l_1/4$ from the center of a column and extending in a rectangular direction toward the column to a point $c/2$ therefrom and thence along a one-quarter circumference about the column center to the adjacent edge of the panel.

[1] The column strip and the middle strip to be used when considering moments in the direction of the dimension l are located and dimensioned as shown in Fig. 15. The dimension l_1 does not always represent the short length of the panel. When moments in the direction of the shorter panel length are considered, the dimensions l and l_1 are to be interchanged and the strips corresponding to those shown in Fig. 15 but extending in the direction of the shorter panel length are to be considered.

[2] These are approximately the values which would be obtained by considering one-third of the total moment, M_0, as positive and two-thirds of it as negative moment.

(*c*) *Section for Positive Moment in Middle Strip:* The section of a length $l_1/2$ extending in a rectangular direction across the center of the middle strip.

(*d*) *Section for Positive Moment in Column Strip:* The section of length $l_1/4$ extending in a rectangular direction across the center of the column strip.

Moments in Principal Design Sections.

144. The moments in the principal design sections shall be those given in Table VI, except as follows:

(*a*) The sum of the maximum negative moments in the two column strips may be greater or less than the values given in Table VI by not more than 0.03 M_0.

(*b*) The maximum negative moment and the maximum positive moments in the middle strip and the sum of the maximum positive moments in the two column strips may each be greater or less than the values given in Table VI by not more than 0.01 M_0.

Thickness of Flat Slabs and Dropped Panels.

145. The total thickness,[1] t_1, of the dropped panel in inches, or of the slab if a dropped panel is not used, shall be not less than:

$$t_1 = 0.038\left(1 - 1.44\frac{c}{l}\right) l \sqrt{Rw'\frac{l_1}{b_1}} + 1\tfrac{1}{2} \ldots\ldots\ldots\ldots (37)^2$$

where R = ratio of negative moment in the two column strips to M_0;

w' = uniformly distributed dead and live load per unit of area of floor; and

b_1 = dimension of the dropped panel in the direction parallel to l_1.

For slabs with dropped panels the total thickness[1] in inches at points beyond the dropped panel shall be not less than

$$t_2 = 0.02\, l\, \sqrt{w'} + 1 \ldots\ldots\ldots\ldots\ldots\ldots (38)$$

The slab thickness t_1 or t_2 shall in no case be less than $l/32$ for floor slabs, and not less than $l/40$ for roof slabs. In determining minimum thickness by Formulas 37 and 38, the value of l shall be the panel length center to center of the columns, on long side of panel, l_1 shall be the panel length on the short side of the panel, and b_1 shall be the width or diameter of dropped panel in the direction of l_1, except that in a slab without dropped panel b_1 shall be 0.5 l_1.

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[1] The thickness will be in inches regardless of whether l and w' are in feet and pounds per square foot or in inches and pounds per square inch.

[2] The values of R used in this formula are the coefficients of M_0 for negative moment in two column strips in Table VI.

146. The dropped panel shall have a length or diameter in each rectangular direction of not less than one-third the panel length in that direction, and a thickness not greater than 1.5 t_2. **Minimum Dimensions of Dropped Panels.**

147. In wall panels and other panels in which the slab is discontinuous at the edge of the panel, the maximum negative moment one panel length away from the discontinuous edge and the maximum positive moment between shall be increased as follows: **Wall and Other Irregular Panels.**

(*a*) Column strip perpendicular to the wall or discontinuous edge, 15 per cent greater than that given in Table VI;

(*b*) Middle strip perpendicular to wall or discontinuous edge, 30 per cent greater than that given in Table VI.

In these strips the bars used for positive moments perpendicular to the discontinuous edge shall extend to the edge of the panel at which the slab is discontinuous.

148. In panels having a marginal beam on one edge or on each of two adjacent edges, the beam shall be designed to carry at least the load superimposed directly upon it, exclusive of the panel load. A beam which has a depth greater than the thickness of the dropped panel into which it frames, shall be designed to carry, in addition to the load superimposed upon it, at least one-fourth of the distributed load for which the adjacent panel or panels are designed, and each column strip adjacent to and parallel with the beam shall be designed to resist a moment at least one-half as great as that specified in Table VI for a column strip.[1] **Panels with Marginal Beams.**

Each column strip adjacent to and parallel with a marginal beam which has a depth less than the thickness of the dropped panel into which it frames shall be designed to resist the moments specified in Table VI for a column strip. Marginal beams on opposite edges of a panel and the slab between them shall be designed for the entire load and the panels shall be designed as simple beams.

149. The negative moments on sections at and parallel to the wall, or discontinuous edge of an interior panel, shall be determined by the conditions of restraint.[2] **Discontinuous Panels.**

150. Where there is a beam or a bearing wall on the center line of columns in the interior portion of a continuous flat slab, the negative moment at the beam or wall line in the middle strip perpendicular to the beam or wall shall be taken as 30 per cent greater than the moment specified in Table VI for a middle strip. The column strip adjacent **Flat Slabs on Bearing Walls.**

[1] In wall columns, brackets are sometimes substituted for capitals or other changes are made in the design of the capital. Attention is directed to the necessity for taking into account the change in the value of c in the moment formula for such cases.

[2] The committee is not prepared to make a more definite recommendation at this time.

to and lying on either side of the beam or wall shall be designed to resist a moment at least one-half of that specified in Table VI for a column strip.

Point of Inflection.

151. The point of inflection in any line parallel to a panel edge in interior panels of symmetrical slabs without dropped panels shall be assumed to be at a distance from the center of the span equal to three-tenths of the distance between the two sections of critical negative moment at opposite ends of the line; for slabs having dropped panels, the coefficient shall be 0.25.

Reinforcement.

152. The reinforcement bars which cross any section and which fulfill the requirements given in Section 153 may be considered as effective in resisting the moment at the section. The sectional area of a bar multiplied by the cosine of the angle between the direction of the axis of the bar and any other direction may be considered effective as reinforcement in that direction.

Arrangement of Reinforcement.

153. The design shall include adequate provision for securing the reinforcement in place so as to take not only the critical moments but the moments at intermediate sections. Provision shall be made for possible shifting of the point of inflection by carrying all bars in rectangular or diagonal directions, each side of a section of critical moment, either positive or negative, to points at least 20 diameters beyond the point of inflection as specified in Section 151. Lapped splices shall not be permitted at or near regions of maximum stress except as described above. At least four-tenths of all bars in each direction shall be of such length and shall be so placed as to provide reinforcement at two sections of critical negative moment and at the intermediate section of critical positive moment. Not less than one-third of the bars used for positive reinforcement in the column strip shall extend into the dropped panel not less than 20 diameters of the bar, or in case no dropped panel is used, shall extend to a point not less than one-eighth of the span length from the center line of the column or the support.

Reinforcement at Construction Joints.

154. See Section 72.

Tensile Stress in Reinforcement.

155. The tensile stress f_s in the reinforcement in flat slabs shall be taken as not less than that computed by Formula 39,

$$f_s = \frac{RM_0}{A_s jd} \qquad (39)$$

where RM_0 = moment specified in Section 144 for two column strips or for one middle strip; and

A_s = effective cross-sectional area of the reinforcement which crosses any of the principal design sections and which meets the requirements of Section 153.

The stress so computed shall not at any of the principal design sections exceed the values specified in Section 194.

156. The compressive stress in the concrete in flat slabs shall be taken as not less than that computed by Formulas 40 and 41, but the stress so computed shall not exceed $0.4f'_c$. **Compressive Stress in Concrete.**

Compression due to negative moment, RM_0, in the two column strips,

$$f_c = \frac{3.5\ RM_0}{0.67\sqrt[3]{pn}\ b_1 d^2}\left(1-1.2\frac{c}{l}\right) \ldots\ldots\ldots\ldots (40)$$

where b_1 is as specified in Section 145.

Compression due to positive moment, RM_0, in the two column strips, or negative or positive moment in the middle strip,

$$f_c = \frac{6\ RM_0}{0.67\sqrt[3]{pn}\ l_1 d^2} \ldots\ldots\ldots\ldots\ldots\ldots (41)$$

In special cases where supported by satisfactory engineering analysis, approved by the Engineer, compression reinforcement may be used to increase the resistance to compression in accordance with other provisions of these specifications.

157. See Section 131. **Shearing Stress.**

158. For structures having a width of one or two panels, and also for slabs having panels of markedly different sizes, an analysis shall be made of the moments developed in both slab and columns, and the values given in Sections 142 to 157 modified accordingly. **Unusual Panels.**

159. See Section 171. **Bending Moments in Columns.**

G. Reinforced Concrete Columns.

160. The following sections on reinforced concrete columns are based on the assumption of a short column. Where the unsupported length is greater than 40 times the least radius of gyration (40 R), the safe load shall be determined by Formula 47. Principal columns in buildings shall have a minimum diameter or thickness of 12 in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of 6 in. **Limiting Dimensions.**

161. The unsupported length of reinforced concrete columns shall be taken as: **Unsupported Length.**

(a) In flat slab construction the clear distance between the floor and under side of the capital;

(*b*) In beam-and-slab construction, the clear distance between the floor and the under side of the shallowest beam framing into the column at the next higher floor level;

(*c*) In floor construction with beams in one direction only, the clear distance between floor slabs;

(*d*) In columns supported laterally by struts or beams only, the clear distance between consecutive pairs (or groups) of struts or beams, provided that to be considered an adequate support, two such struts or beams shall meet the column at approximately the same level and the angle between the two planes formed by the axis of the column and the axis of each strut respectively is not less than 75 deg. nor more than 105 deg.

When haunches are used at the junction of beams or struts with columns, the clear distance between supports may be considered as reduced by two-thirds of the depth of the haunch.

Safe Load on Spiral Columns.

162. The safe axial load on columns reinforced with longitudinal bars and closely spaced spirals enclosing a circular core shall be not greater than that determined by Formula 42.

The symbols used in Formulas 42 to 49 are defined in Section 105, except as indicated in Sections 162, 165, 168, 170, 176 and 182.

$$P = A_c f_c + n f_c pA \qquad (42)$$

where

P = total safe axial load on column whose h/R is less than 40;

A = area of the concrete core enclosed within the spiral; the diameter of the core (or of the spiral) shall be taken as the distance center to center of the spiral wire;

p = ratio of effective area of longitudinal reinforcement to area of the concrete core;

$A_c = A\ (1-p)$ = net area of concrete core; and

f_c = permissible compressive stress in concrete =

$$300 + (0.10 + 4p)\, f'_c \qquad (43)$$

The longitudinal reinforcement shall consist of at least six bars of minimum diameter of $\frac{1}{2}$ in., and its effective cross-sectional area shall not be less than 1 per cent nor more than 6 per cent of that of the core.

Spiral Reinforcement.

163. The spiral reinforcement shall be not less than one-fourth the volume of the longitudinal reinforcement. It shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. The spacing of the spirals shall be not greater than one-sixth of the diameter of the core and in

no case more than 3 in. The spiral reinforcement shall meet the requirements of the Tentative Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement. (Appendix 7.)

164. Reinforcement shall be protected everywhere by a covering of concrete cast monolithic with the core, which shall have a minimum thickness of $1\frac{1}{2}$ in. in square columns and 2 in. in round or octagonal columns. **Protection of Spirally Reinforced Column.**

165. The safe axial load on columns reinforced with longitudinal bars and separate lateral ties shall be not greater than that determined by Formula 44, **Safe Load on Columns with Lateral Ties.**

$$P = (A'_c + A_s n) f_c \ldots\ldots\ldots\ldots\ldots\ldots (44)$$

where A'_c = net area of concrete in the column (total column area minus area of reinforcement);

A_s = effective cross-sectional area of longitudinal reinforcement; and

f_c = permissible compressive stress in concrete and shall not exceed $0.20f'_c$.

The amount of longitudinal reinforcement considered in the calculations shall be not more than 2 per cent nor less than 0.5 per cent of the total area of the column. The longitudinal reinforcement shall consist of not less than four bars of minimum diameter of $\frac{1}{2}$ in., placed with clear distance from the face of the column not less than 2 in.

166. Lateral ties shall be not less than $\frac{1}{4}$ in. in diameter, spaced not more than 8 in. apart. **Lateral Ties.**

167. Reinforced concrete columns subject to bending stresses shall be treated as follows: **Bending in Columns.**

(*a*) *With Spiral Reinforcement.*—The compressive unit stress on the concrete within the core area under combined axial load and bending shall not exceed by more than 20 per cent the value given for axial load by Formula 43.

(*b*) *With Lateral Ties.*—Additional longitudinal reinforcement may be used if required and the compressive unit stress on the concrete under combined axial load and bending may be increased to $0.30 f'_c$. The total amount of reinforcement considered in the computations shall be not more than 4 per cent of the total area of the column.

Tension in the longitudinal reinforcement due to bending of the column shall not exceed 16,000 lb. per sq. in.

168. The safe load on composite columns in which a structural steel or cast-iron column is thoroughly encased in a circumferentially reinforced concrete core shall be based on a certain unit stress for **Composite Columns.**

the steel or cast-iron core plus a unit stress of $0.25 f'_c$ on the area within the spiral core.

The unit compressive stress on the steel section shall be not greater than that determined by Formula 45,

$$f_r = 18{,}000 - 70\, h/R \qquad (45)$$

but shall not exceed 16,000 lb. per sq. in.

The unit stress on the cast-iron section shall be not greater than that determined by Formula 46,

$$f_r = 12{,}000 - 60\, h/R \qquad (46)$$

but shall not exceed 10,000 lb. per sq. in.

In Formulas 45 and 46,

f_r = compressive unit stress in metal core, and

R = least radius of gyration of the steel or cast-iron section.

The diameter of the cast-iron section shall not exceed one-half of the diameter of the core within the spiral. The spiral reinforcement shall be not less than 0.5 per cent of the volume of the core within the spiral and shall conform in quality, spacing and other requirements to the provisions for spirals in Section 163.

Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders. The area of the concrete between the spiral and the metal core shall be not less than that required to carry the total floor load of the story above on the basis of a stress in the concrete of $0.35 f'_c$, unless special brackets are arranged on the metal core to receive directly the beam or slab load.

Structural Steel Columns. 169. The safe load on a structural steel column of a section which fully encases an area of concrete, and which is protected by an outside shell of concrete at least 3 in. thick, shall be computed in the same manner as for composite columns in Section 168, allowing $0.25 f'_c$ on the area of the concrete enclosed by the steel section. The outside shell shall be reinforced by wire mesh, ties or spiral hoops weighing not less than 0.2 lb. per sq. ft. at the surface of the mesh and with a maximum spacing of 6 in. between strands or hoops. Special brackets shall be used to receive the entire floor load at each story. The safe load in steel columns calculated by Formula 45 shall not exceed 16,000 lb. per sq. in.

Long Columns. 170. The permissible working load on the core in axially loaded columns which have a length greater than 40 times the least radius of gyration of the column core (40 R) shall be not greater than that determined by Formula 47,

$$\frac{P'}{P} = 1.33 - \frac{h}{120\, R} \qquad (47)$$

where P' = total safe axial load on long column;
P = total safe axial load on column of the same section whose h/R is less than 40, determined as in Sections 162 and 165; and
R = least radius of gyration of column core.

171. The bending moments in interior and exterior columns shall be determined on the basis of loading conditions and end restraint, and shall be provided for in the design. The recognized methods shall be followed in calculating the stresses due to combined axial load and bending. In spiral columns the area to be considered as resisting the stress is the area within the spiral. **Bending Moments in Columns.**

H. Footings.

172. The requirements for tension, compression, shear and bond in Sections 103 and 141, inclusive, shall govern the design of footings, except as hereinafter provided. **General.**

173. The load per unit of area on soil footings shall be computed by dividing the column load by the area of base of the footing. **Soil Footings.**

174. Footings on piles shall be treated in the same manner as footings on soil, except that the load shall be considered as concentrated at the pile centers. **Pile Footings.**

175. Footings in which the thickness has been determined by the requirements for shear as specified in Sections 133 and 134 may be sloped or stepped between the critical section and the edge of the footing, provided that the shear on no section outside the critical section exceeds the value specified, and provided further that the thickness of the footing above the reinforcement at the edge shall not be less than 6 in. for footings on soil nor less than 12 in. for footings on piles. Sloped or stepped footings shall be cast as a unit. **Sloped or Stepped Footings.**

176. The critical section for bending in a concrete footing which supports a concrete column or pedestal, shall be considered to be at the face of the column or pedestal. Where steel or cast-iron column bases are used, the moment in the footing shall be computed at the middle and at the edge of the base; the load shall be considered as uniformly distributed over the column or pedestal base. **Critical Section for Bending.**

The bending moment at the critical section in a square footing supporting a concentric square column, shall be computed from the load on the trapezoid bounded by one face of the column, the corresponding outside edge of the footing, and the portions of the two diagonals. The load on the two corner triangles of this trapezoid shall be considered as applied at a distance from the face equal to

six-tenths of the projection of the footing from the face of the column. The load on the rectangular portion of the trapezoid shall be considered as applied at its center of gravity. The bending moment is expressed by Formula 48,

$$M = \frac{w}{2}(a+1.2\,c)c^2 \qquad (48)$$

where M = bending moment at critical section of footing;
a = width of face of column or pedestal;
c = projection of footing from face of column; and
w = upward reaction per unit of area of base of footing.

For a round or octagonal column, the distance a shall be taken as equal to the side of a square of an area equal to the area enclosed within the perimeter of the column. (For typical footing designs, see Appendix 1, Figs. 16 and 17.)

Reinforcement. 177. The reinforcement in each direction in the footing shall be determined as for a reinforced concrete beam; the effective depth shall be the distance from the top of the footing to the plane of the reinforcement. The sectional area of reinforcement shall be distributed uniformly across the footing unless the width is greater than the side of the column or pedestal plus twice the effective depth of the footing, in which case the width over which the reinforcement is spread may be increased to include one-half the remaining width of the footing. In order that no considerable area of the footing shall remain unreinforced, additional reinforcement shall be placed outside of the width specified, but such reinforcement shall not be considered as effective in resisting the calculated bending moment. For the extra reinforcement a spacing double that within the effective belt may be used.

Concrete Stress. 178. The extreme fiber stress in compression in the concrete shall be kept within the limits specified in Section 189. The extreme fiber stress in sloped or stepped footings shall be based on the exact shape of the section for a width not greater than that assumed effective for reinforcement.

Irregular Footings. 179. A rectangular or irregularly shaped footing shall be computed by dividing it into rectangles or trapezoids tributary to the sides of the column, using the distance to the center of gravity of the area as the moment arm of the upward forces. Outstanding portions of combined footings shall be treated in the same manner. Other portions of combined footings shall be designed as beams or slabs.

Shearing Stresses. 180. See Sections 132 to 134.

181. See Sections 135 to 141. **Bond Stress.**

182. The compressive stress in longitudinal reinforcement at the base of a column shall be transferred to the pedestal or footing by either dowels or distributing bases. When dowels are used, there shall be at least one for each column bar, and the total sectional area of the dowels shall be not less than the sectional area of the longitudinal reinforcement in the column. The dowels shall extend into the column and into the pedestal or footing not less than 50 diameters of the dowel bars for plain bars, or 40 diameters for deformed bars. **Transfer of Stress at Base of Column.**

When metal distributing bases are used, they shall have sufficient area and thickness to transmit safely the load from the longitudinal reinforcement in compression and bending. The permissible compressive unit stress on top of the pedestal or footing directly under the column shall be not greater than that determined by Formula 49,

$$r_a = 0.25\, f'_c \sqrt[3]{\frac{A}{A'}} \quad \ldots\ldots\ldots\ldots (49)$$

where r_a = permissible working stress over the loaded area;

A = total area at the top of the pedestal or footing;

A' = loaded area at the column base;

f'_c = ultimate compressive strength of concrete. (See Section 120.)

In sloped or stepped footings A may be taken as the area of the top horizontal surface of the footing or as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the loaded area A', and having side slopes of 1 vertical to 2 horizontal.

183. The allowable compressive unit stress on the gross area of a concentrically loaded pedestal or on the minimum area of a pedestal footing shall not exceed $0.25\, f'_c$, unless reinforcement is provided and the member designed as a reinforced concrete column. **Pedestals Without Reinforcement.**

The depth of a pedestal or pedestal footing shall be not greater than three times its least width and the projection on any side from the face of the supported member shall be not greater than one-half the depth. The depth of a pedestal whose sides are sloped or stepped shall not exceed three times the least width or diameter of the section midway between the top and bottom. A pedestal footing supported directly on piles shall have a mat of reinforcing bars having a cross-sectional area of not less than 0.20 sq. in. per foot in each direction, placed 3 in. above the top of the piles.

I. *Reinforced Concrete Retaining Walls.*

Loads and Unit Stresses.

184. Reinforced concrete retaining walls shall be so designed[1] that the permissible unit stresses specified in Sections 186 to 197 are not exceeded. The heels of cantilever, counterforted and buttressed retaining walls shall be proportioned for maximum resultant vertical loads, but when the foundation reaction is neglected the permissible unit stresses shall not be more than 50 per cent greater than the normal permissible stresses.

Details of Design.

185. The following principles shall be followed in the design of reinforced concrete retaining walls:

(*a*) The unsupported toe and heel of the base slabs shall be considered as cantilever beams fixed at the edge of the support.

(*b*) The vertical section of a cantilever wal shall be considered as a cantilever beam fixed at the top of the base.

(*c*) The vertical sections of counterforted and buttressed walls and parts of base slabs supported by the counterforts or buttresses shall be designed in accordance with the requirements for a continuous slab in Section 110.

(*d*) The exposed faces of walls without buttresses shall preferably be given a batter of not less than $\frac{1}{4}$ in. per ft.

(*e*) Counterforts shall be designed in accordance with the requirements for T-beams in Sections 113 to 115. Stirrups shall be provided in the counterforts to take the reaction when the tension reinforcement of the face walls and heels of bases is designed to span between the counterforts. Stirrups shall be anchored as near the exposed face of the longitudinal wall and as close to the lower face of the base as the requirements for protective covering permit.

(*f*) Buttresses shall be designed in accordance with the requirements specified for rectangular beams.

(*g*) The shearing stress at the junction of the base with counterforts or buttresses shall not exceed the values specified in Sections 120 to 130.

(*h*) Horizontal metal reinforcement shall be of such form and so distributed as to develop the required bond. To prevent temperature and shrinkage cracks in exposed surface not less than 0.25 sq. in. of horizontal metal reinforcement per foot of height shall be provided.

(*i*) Grooved lock joints shall be placed not over 60 ft. apart to care for temperature changes.

(*j*) Counterforts and buttresses shall be located under all points

[1] In proportioning retaining walls consideration should be given to:
(*a*) Bearing value of soil;
(*b*) Stability against sliding.

of concentrated loading, and at intermediate points, as may be required by the design.

(*k*) The walls shall be cast as a unit between expansion joints, unless construction joints formed in accordance with Sections 69 and 73 are provided.

(*l*) Drains or "weep holes" not less than 4 in. in diameter and not more than 10 ft. apart, shall be provided. At least one drain shall be provided for each pocket formed by counterforts.

J. Summary of Working Stresses.

186. The following working stresses shall be used: **General.**
where, f'_c = ultimate compressive strength of concrete at age of 28 days, based on tests of 6 by 12-in. or 8 by 16-in. cylinders made and tested in accordance with the Standard Methods of Making and Storing Specimens of Concrete in the Field (Appendix 13) and the Tentative Methods of Making Compression Tests of Concrete (Appendix 12).

Direct Stress in Concrete.

187. (*a*) Columns whose length does not exceed $40R$: **Direct Compression.**
(1) With spirals .... varies with amount of longitudinal reinforcement. (See Section 162.)
(2) Longitudinal reinforcement and lateral ties. (See Section 165.)
(*b*) Long columns ....(See Section 170.)
(*c*) Piers and Pedestals.............................. $0.25f'_c$
(See Section 183.)

188. (*a*) Extreme fiber stress in flexure...................... $0.40f'_c$ **Compression in Extreme Fiber.**
(*b*) Extreme fiber stress in flexure adjacent to supports of continuous beams.......................... $0.45f'_c$

189. In Concrete Members..................................None **Tension.**

Shearing Stresses in Concrete.

190. (*a*) Beams without web reinforcement................. $0.02f'_c$ **Longitudinal Bars without Special Anchorage.**
(*b*) Beams with stirrups or bent-up bars or combination of the two............................ $0.06f'_c$

191. (*a*) Beams without web reinforcement................. $0.03f'_c$ **Longitudinal Bars Having Special Anchorage.**
(*b*) Beams with stirrups or bent-up bars or a combination of the two............................ $0.12f'_c$

192. (*a*) Shear at distance *d* from capital or dropped panel... $0.03f'_c$ **Flat Slabs.**
(*b*) Other limiting cases in flat slabs. (See Section 131.)

Footings. 193. (*a*) Longitudinal bars without special anchorage........ $0.02f'_c$
(*b*) Longitudinal bars having special anchorage......... $0.03f'_c$

Stresses in Reinforcement.

Tension in Steel. 194. (*a*) Billet-steel bars:
(1) Structural steel grade............ 16,000 lb. per sq. in.
(2) Intermediate grade.............. 18,000 " "
(3) Hard grade.................... 18,000 " "
(*b*) Rail-steel bars...................... 18,000 " "
(*c*) Structural steel..................... 16,000 " "
(*d*) Cold-drawn steel wire:
(1) Spirals..........................Stress not calculated
(2) Elsewhere...................... 18,000 lb. per sq. in.

Compression in Steel. 195. (*a*) Bars......................same as Section 194 (*a*) and (*b*)
(*b*) Structural steel core of composite column...................... 16,000 lb. per sq. in.
reduced for slenderness ratio..........(see Section 168)
(*c*) Structural steel column.............. 16,000 lb. per sq. in.
reduced for slenderness ratio..........(see Section 169)

Compression in Cast Iron. 196. Composite cast-iron column .... 10,000 lb. per sq. in.
reduced for slenderness ratio..........(see Section 168)

Bond Between Concrete and Reinforcement. 197. (*a*) Beams and slabs, plain bars...................... $0.04f'_c$
(*b*) Beams and slabs, deformed bars.................. $0.05f'_c$
(*c*) Footings, plain bars, one-way.................... $0.04f'_c$
(*d*) Footings, deformed bars, one way................. $0.05f'_c$
(*e*) Footings, bars two ways....(*c*) or (*d*) reduced by 25 per cent

APPENDIX 1

NOTATIONS AND FIGURES

All symbols used in the Standard Specifications for Concrete and Reinforced Concrete have been collected here for convenience of reference. The symbols are in general defined in the text near the formulas in which they are used. In a few instances the same symbol is used in two distinct senses; however, there is little danger of confusion from this source.

NOTATION

a = width of face of column or pedestal;

α = angle between inclined web bars and longitudinal bars;

A = total net area of column, footing, or pedestal, exclusive of fireproofing;

A' = loaded area of pedestal, pier or footing;

A_c = $A(1-p)$ = net area of concrete core of column (core area minus reinforcement);

A'_c = net area of concrete in columns with lateral ties (total column area minus area of reinforcement);

A_s = effective cross-sectional area of metal reinforcement in tension in beams or compression in columns; and the effective cross-sectional area of metal reinforcement which crosses any of the principal design sections of a flat slab and which meets the requirements of Section 153;

A_v = total area of web reinforcement in tension within a distance of s (s_1, s_2, s_3, etc.) or the total area of all bars bent up in any one plane (see Fig. 9);

b = width of rectangular beam or width of flange of T-beam;

b' = width of stem of T-beam;

b_1 = dimension of the dropped panel of a flat slab in the direction parallel to l_1;[1]

c = base diameter of the largest right circular cone which lies entirely within the column (including the capital) whose vertex angle is 90 deg. and whose base is $1\frac{1}{2}$ in. below the bottom of the slab or the bottom of the dropped panel (see Fig. 14);

c = projection of footing from face of column;

C = total compressive stress in concrete;

[1] In flat slab design, the column strip and the middle strip to be used when considering moments in the direction of the dimension l are located and dimensioned as shown in Fig. 15. The dimension l_1 does not always represent the short length of the panel. When moments in the direction of the shorter panel length are considered, the dimensions l and l_1 are to be interchanged and strips corresponding to those shown in Fig. 15 but extending in the direction of the shorter panel length are to be considered.

C' = total compressive stress in reinforcement;
d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement;
d' = depth from compression surface of beam or slab to center of compression reinforcement;
E_c = modulus of elasticity of concrete in compression;
E_s = modulus of elasticity of steel in tension = 30,000,000 lb. per sq. in.;
f_c = compressive unit stress in extreme fiber of concrete;
f'_c = ultimate compressive strength of concrete at age of 28 days, based on tests of 6 by 12-in. or 8 by 16-in. cylinders made and tested in accordance with the Standard Methods of Making and Storing Specimens of Concrete in the Field (Appendix 13) and the Tentative Methods of Making Compression Tests of Concrete (Appendix 12);
f_r = compressive unit stress in metal core;
f_s = tensile unit stress in longitudinal reinforcement;
f'_s = compressive unit stress in longitudinal reinforcement;
f_v = tensile unit stress in web reinforcement;
F = total tension in a bar;
h = unsupported length of column;
I = moment of inertia of a section about the neutral axis for bending;
j = ratio of lever arm of resisting couple to depth d;
jd = $d-z$ = arm of resisting couple;
k = ratio of depth of neutral axis to depth d;
l = span length of beam or slab (generally distance from center to center of supports; for special cases, see Sections 106 and 145);
l = span length of flat slab, center to center of columns, in the rectangular direction in which moments are considered;[1]
l_1 = span length of flat slab, center to center of columns, perpendicular to the rectangular direction in which moments are considered;[1]
M = bending moment or moment of resistance in general;
M_0 = sum of positive and negative bending moments in either rectangular direction, at the principal design sections of a panel of a flat slab;
n = E_s/E_c = ratio of modulus of elasticity of steel to that of concrete;
Σo = sum of perimeters of bars in one set;
p = ratio of effective area of tension reinforcement to effective

[1] See footnote regarding b_1 in foregoing notation, p. 67.

area of concrete in beams $= A_s/bd$; and the ratio of effective area of longitudinal reinforcement to the area of the concrete core in columns;

p' = ratio of effective area of compression reinforcement to effective area of concrete in beams;

P = total safe axial load on column whose h/R is less than 40;

P' = total safe axial load on long column;

Q = ratio of the average to the maximum bond stress computed by Formula 34 within the distance y;

r = ratio of cross-sectional area of negative reinforcement which crosses entirely over the column capital of a flat slab or over the dropped panel, to the total cross-sectional area of the negative reinforcement in the two column strips;

r_a = permissible working stress in concrete over the loaded area of a pedestal, pier or footing;

R = ratio of positive or negative moment in two column strips or one middle strip of a flat slab, to M_0;

R = least radius of gyration of a section;

s = spacing of web members, measured at the plane of the lower reinforcement and in the direction of the longitudinal axis of the beam;

t = thickness of flange of T-beam;

t_1 = thickness of flat slab without dropped panels or thickness of a dropped panel (see Fig. 14);

t_2 = thickness of flat slab with dropped panels at points away from the dropped panel (see Fig 14);

T = total tensile stress in longitudinal reinforcement;

u = bond stress per unit of area of surface of bar;

v = shearing unit stress;

V = total shear;

w = uniformly distributed load per unit of length of beam or slab;

w = upward reaction per unit of area of base of footing;

w' = uniformly distributed dead and live load per unit of area of a floor or roof;

W = total dead and live load uniformly distributed over a single panel area;

x = length of bar added for anchorage, including the hook, if any;

y = distance from the point at which the tension is computed to the point of beginning of anchorage;

z = depth from compression surface of beam or slab to resultant of compressive stresses.

FIGURES.

For explanation of symbols used in figures, see foregoing notation.

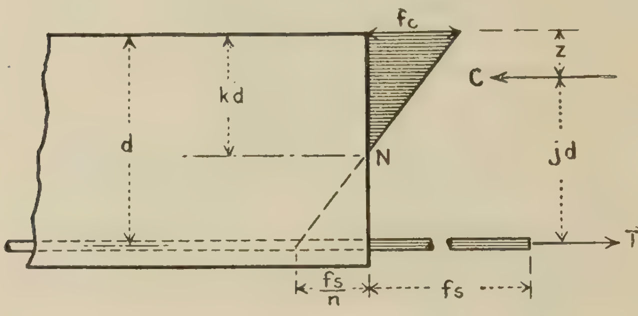

FIG. 1.—Nomenclature for Concrete Beam Reinforced for Tension.

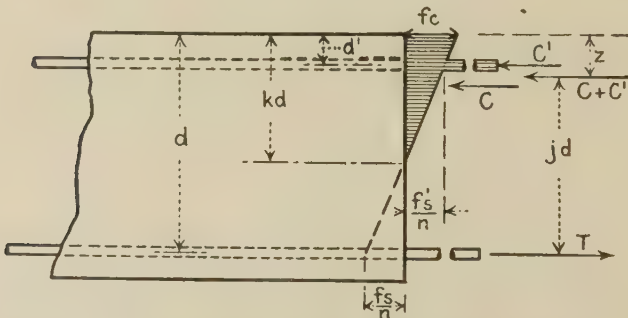

FIG. 2.—Nomenclature for Concrete Beam Reinforced for Tension and Compression.

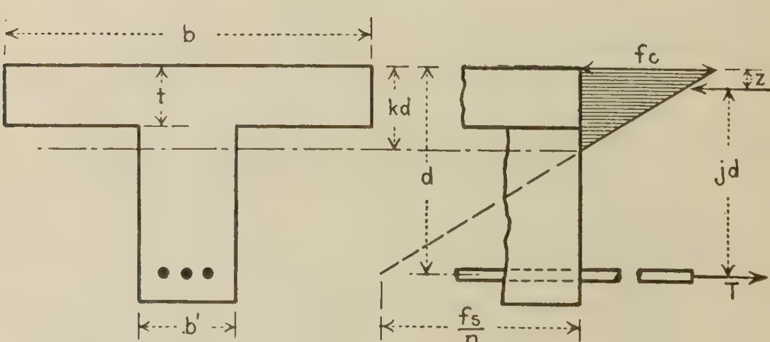

FIG. 3.—Nomenclature for Reinforced Concrete T-Beam.

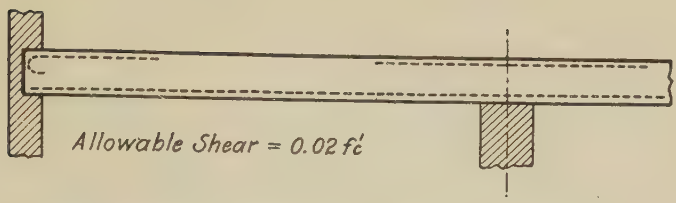

FIG. 4.—Typical Reinforced Concrete Beam; Principal Longitudinal Bars without Special Anchorage.

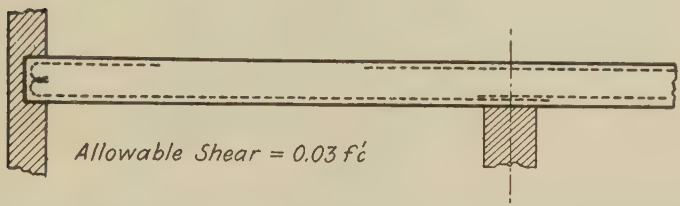

FIG. 5.—Typical Reinforced Concrete Beam; Special Anchorage of Longitudinal Bars.

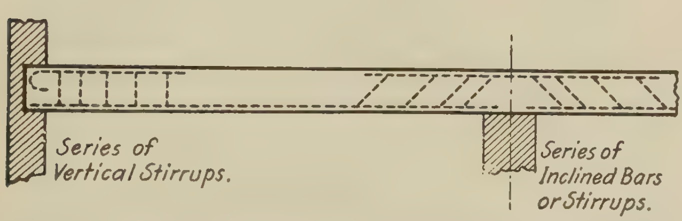

FIG. 6.—Typical Reinforced Concrete Beam without Special Anchorage; Web Reinforced by Means of Series of Vertical Stirrups or Series of Inclined Bars or Stirrups.

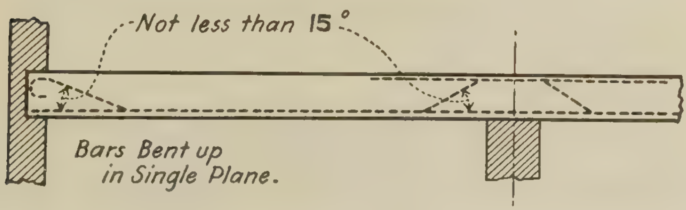

FIG. 7.—Typical Reinforced Concrete Beam; Principal Longitudinal Bars Bent up in Single Plane.

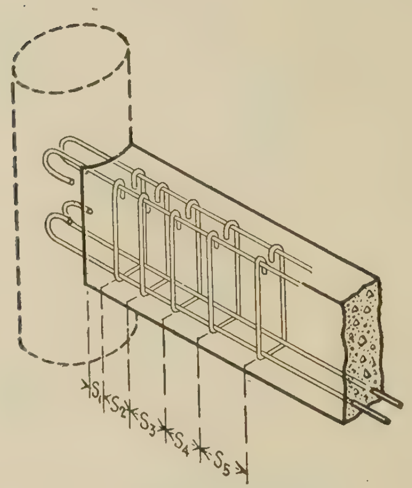

FIG. 8.—Typical Reinforced Concrete Beam with Anchored Longitudinal Bars and Vertical Stirrups.

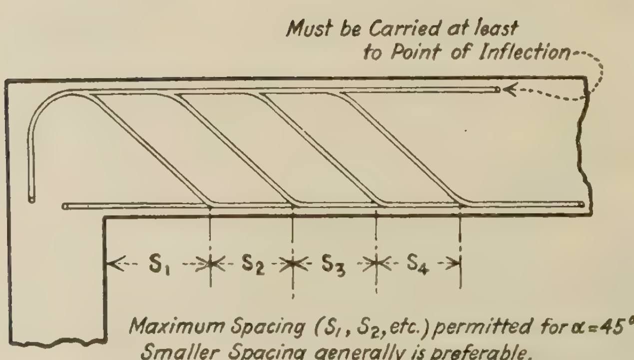

FIG. 9.—Typical Beam with Web Reinforced by Means of Series of Inclined Bars.

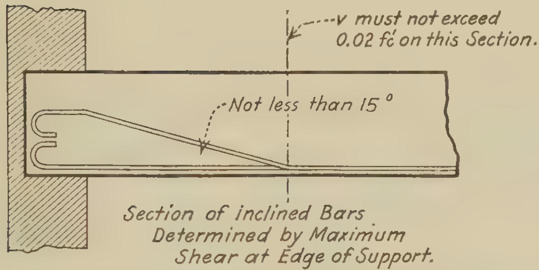

FIG. 10.—Typical Beam with Web Reinforced by Means of Bars Bent up in Single Plane.

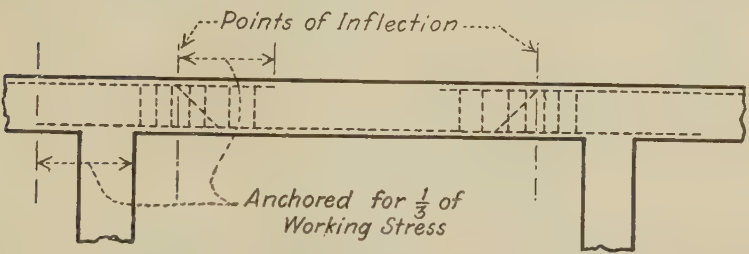

FIG. 11.—Typical Web Reinforcement for Continuous Beams with Special Anchorages.

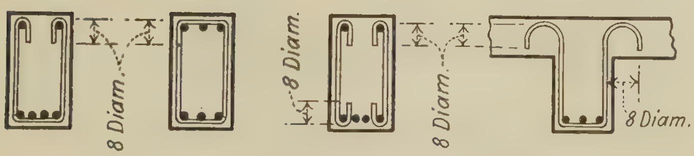

FIG. 12.—Typical Methods of Anchoring Vertical Stirrups.

FIG. 13.—Typical Reinforced Concrete Beam-and-Tile Construction.

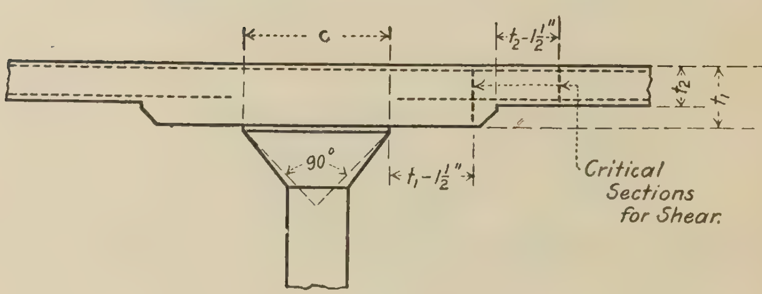

FIG. 14.—Typical Column Capital and Sections of Flat Slab with Dropped Panel.

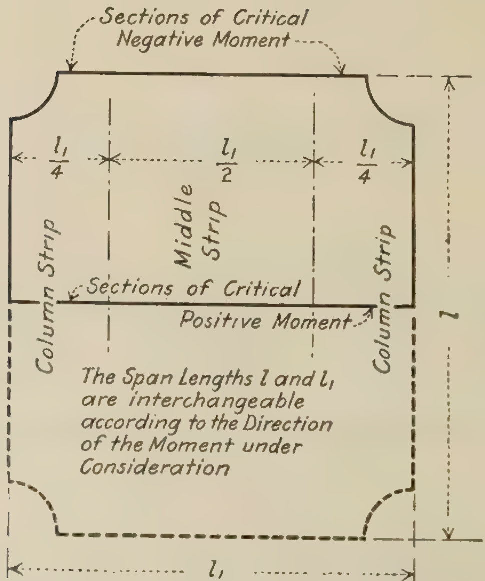

FIG. 15.—Principal Design Sections of a Flat Slab.

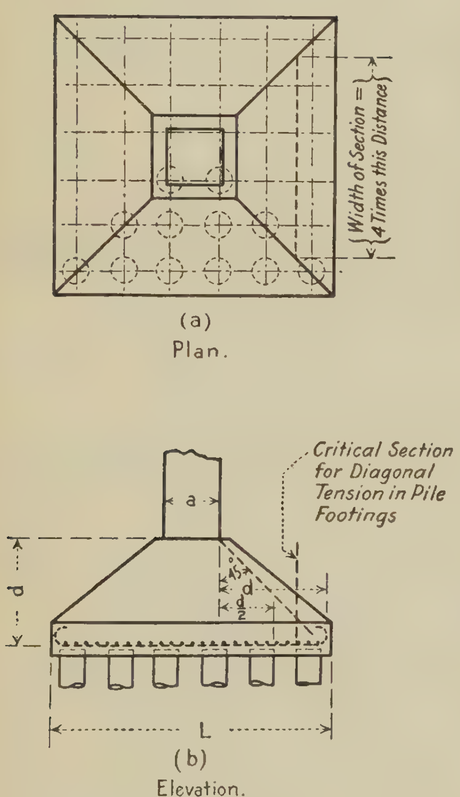

Fig. 16.—Typical Sloped Reinforced Concrete Footing on Piles.

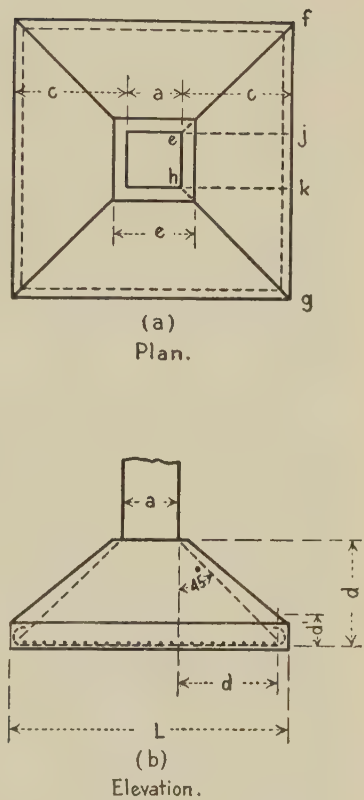

Fig. 17.—Typical Sloped Reinforced Concrete Footing on Soil.

APPENDIX 16.

PROPORTIONS[1] FOR CONCRETE OF GIVEN COMPRESSIVE STRENGTH AT 28 DAYS.

The table gives the proportions in which portland cement and a wide range in sizes of fine and coarse aggregates should be mixed to obtain concrete of compressive strengths ranging from 1500 to 3000 lb. per sq. in. at 28 days. Proportions are given for concrete of four different consistencies.

The purpose of the table is twofold:

(1) To furnish a guide in the selection of mixtures to be used in preliminary investigations of the strength of concrete from given materials.

(2) To indicate proportions which may be expected to produce concrete of a given strength under average conditions where control tests are not made.

If the proportions to be used in the work are selected from the table without preliminary tests of the materials, and control tests are not made during the progress of the work, the mixtures in bold-face type shall be used.

The use of this table as a guide in the selection of concrete mixtures is based on the following:

(1) Concrete shall be plastic;

(2) Aggregates shall be clean and structurally sound;

(3) Aggregates shall be graded between the sizes indicated;

(4) Cement shall conform to the requirements of the Standard Specifications and Tests for Portland Cement (Serial Designation: C 9–21) of the American Society for Testing Materials. (Appendix 2.)

The plasticity of the concrete shall be determined by the slump test carried out in accordance with the Tentative Method of Test for Consistency of Portland-Cement Concrete for Pavements or for Pavement Base (Serial Designation: D 138–22 T) of the American Society for Testing Materials. (Appendix 11.)

Apply the following rules in determining the size assigned to a given aggregate:

(1) Not less than 15 per cent shall be retained between the sieve which is considered the maximum size[2] and the next smaller sieve.

(2) Not more than 15 per cent of a coarse aggregate shall be finer than the sieve considered as the minimum size.[2]

(3) Only the sieve sizes given in the table shall be considered in applying rules (1) and (2).

(4) Sieve analysis shall be made in accordance with the Standard Method of Test for Sieve Analysis of Aggregates for Concrete (Serial Designation: C 41–24) of the American Society for Testing Materials. (Appendix 8.)

Proportions may be interpolated for concrete strengths, aggregate sizes and consistencies not covered by the table or determined by test.

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[1] Based on the 28-day compressive strengths of 6 by 12-in. cylinders, made and stored in accordance with the Tentative Methods of Making Compression Tests of Concrete (Serial Designation: C 39–21 T) of the American Society for Testing Materials. (Appendix 12.)

[2] For example: a graded sand with 16 per cent retained on the No. 8 sieve would fall in the 0-No. 4 size; if 14 per cent or less were retained, the sand would fall in the 0-No. 8 size. A coarse aggregate having 16 per cent coarser than 2-in. sieve would be considered as 3-in. aggregate.

PROPORTIONS FOR 1500 LB. PER SQ. IN. CONCRETE.

Proportions are expressed by volume as follows: Portland Cement : Fine Aggregate : Coarse Aggregate.

Thus 1 : 2.6 : 4.6 indicates 1 part by volume of portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate.

Size of Coarse Aggregate.	Slump, in.	Size of Fine Aggregate.				
			0 – No. 14	0 – No. 8	0 – No. 4	0 – $\frac{3}{8}$ in.
None..............	$\frac{1}{2}$ to 1	1 : 2.8	1 : 3.2	1 : 3.8	1 : 4.4	1 : 5.1
	3 " 4	1 : 2.4	1 : 2.8	1 : 3.3	1 : 3.8	1 : 4.5
	6 " 7	**1 : 1.9**	**1 : 2.2**	**1 : 2.6**	**1 : 3.0**	**1 : 3.6**
	8 " 10	1 : 1.4	1 : 1.6	1 : 1.8	1 : 2.1	1 : 2.5
No. 4 to $\frac{3}{4}$ in.......	$\frac{1}{2}$ to 1	1 : 2.6 : 4.6	1 : 2.9 : 4.3	1 : 3.4 : 4.1	1 : 3.9 : 3.6	1 : 4.6 : 3.1
	3 " 4	1 : 2.3 : 4.0	1 : 2.6 : 3.8	1 : 2.9 : 3.6	1 : 3.4 : 3.2	1 : 4.1 : 2.8
	6 " 7	**1 : 1.8 : 3.4**	**1 : 2.0 : 3.2**	**1 : 2.3 : 3.1**	**1 : 2.6 : 2.8**	**1 : 3.1 : 2.5**
	8 " 10	1 : 1.1 : 2.5	1 : 1.3 : 2.4	1 : 1.5 : 2.4	1 : 1.7 : 2.2	1 : 2.1 : 2.0
No. 4 to 1 in.......	$\frac{1}{2}$ to 1	1 : 2.4 : 5.3	1 : 2.7 : 5.2	1 : 3.1 : 5.0	1 : 3.5 : 4.7	1 : 4 3 : 4.3
	3 " 4	1 : 2.1 : 4.7	1 : 2.4 : 4.5	1 : 2.7 : 4.4	1 : 3.1 : 4.1	1 : 3.7 : 3.7
	6 " 7	**1 : 1.6 : 3.9**	**1 : 1.8 : 3.8**	**1 : 2.1 : 3.7**	**1 : 2.4 : 3.5**	**1 : 2.9 : 3.3**
	8 " 10	1 : 1.1 : 2.9	1 : 1.2 : 2.8	1 : 1.4 : 2.8	1 : 1.6 : 2.7	1 : 1.9 : 2.5
No. 4 to 1$\frac{1}{2}$ in......	$\frac{1}{2}$ to 1	1 : 2.4 : 6.0	1 : 2.7 : 5.9	1 : 3.1 : 5.8	1 : 3.5 : 5.4	1 : 4.1 : 5.1
	3 " 4	1 : 2.0 : 5.4	1 : 2.3 : 5.3	1 : 2.7 : 5.2	1 : 3.0 : 5.0	1 : 3.5 : 4.6
	6 " 7	**1 : 1.6 : 4.4**	**1 : 1.8 : 4.3**	**1 : 2.0 : 4.3**	**1 : 2.3 : 4.1**	**1 : 2.7 : 3.9**
	8 " 10	1 : 1.0 : 3.3	1 : 1.1 : 3.2	1 : 1.3 : 3.2	1 : 1.5 : 3.1	1 : 1.8 : 2.9
No. 4 to 2 in.......	$\frac{1}{2}$ to 1	1 : 2.2 : 6.9	1 : 2.4 : 6.8	1 : 2.8 : 6.8	1 : 3.1 : 6.6	1 : 3.7 : 6.4
	3 " 4	1 : 1.8 : 6.2	1 : 2.0 : 6.1	1 : 2.4 : 6.1	1 : 2.7 : 6.0	1 : 3.1 : 5.7
	6 " 7	**1 : 1.4 : 5.1**	**1 : 1.6 : 5.0**	**1 : 1.8 : 5.0**	**1 : 2.0 : 5.0**	**1 : 2.4 : 4.8**
	8 " 10	1 : 0.9 : 3.8	1 : 1.0 : 3.8	1 : 1.1 : 3.8	1 : 1.3 : 3.8	1 : 1.5 : 3.7
$\frac{3}{8}$ to 1 in...........	$\frac{1}{2}$ to 1	1 : 2.8 : 5.2	1 : 3.1 : 5.1	1 : 3.6 : 4.8	1 : 4.2 : 4.6	1 : 4.8 : 4.1
	3 " 4	1 : 2.4 : 4.5	1 : 2.6 : 4.5	1 : 3.1 : 4.3	1 : 3.6 : 4.0	1 : 4.1 : 3.6
	6 " 7	**1 : 1.9 : 3.9**	**1 : 2.1 : 3.7**	**1 : 2.4 : 3.6**	**1 : 2.8 : 3.4**	**1 : 3.2 : 3.1**
	8 " 10	1 : 1.3 : 2.8	1 : 1.4 : 2.8	1 : 1.6 : 2.7	1 : 1.9 : 2.6	1 : 2.2 : 2.4
$\frac{3}{8}$ to 1$\frac{1}{2}$ in.........	$\frac{1}{2}$ to 1	1 : 2.8 : 5.8	1 : 3.1 : 5.7	1 : 3.5 : 5.5	1 : 4.1 : 5.3	1 : 4.7 : 4.9
	3 " 4	1 : 2.4 : 5.2	1 : 2.7 : 5.1	1 : 3.1 : 5.0	1 : 3.5 : 4.8	1 : 4.1 : 4.4
	6 " 7	**1 : 1.9 : 4.3**	**1 : 2.1 : 4.2**	**1 : 2.4 : 4.2**	**1 : 2.7 : 4.0**	**1 : 3.1 : 3.7**
	8 " 10	1 : 1.2 : 3.2	1 : 1.4 : 3.2	1 : 1.6 : 3.1	1 : 1.8 : 3.0	1 : 2.1 : 2.9
$\frac{3}{8}$ to 2 in...........	$\frac{1}{2}$ to 1	1 : 2.7 : 6.6	1 : 3.0 : 6.6	1 : 3.4 : 6.5	1 : 3.9 : 6.4	1 : 4.4 : 6.0
	3 " 4	1 : 2.3 : 5.9	1 : 2.6 : 5.9	1 : 2.9 : 5.8	1 : 3.3 : 5.6	1 : 3.7 : 5.5
	6 " 7	**1 : 1.8 : 4.9**	**1 : 2.0 : 4.8**	**1 : 2.2 : 4.8**	**1 : 2.6 : 4.8**	**1 : 3.0 : 4.5**
	8 " 10	1 : 1.2 : 3.7	1 : 1.3 : 3.7	1 : 1.5 : 3.7	1 : 1.7 : 3.6	1 : 1.9 : 3.5
$\frac{3}{4}$ to 1$\frac{1}{2}$ in.........	$\frac{1}{2}$ to 1	1 : 3.2 : 5.4	1 : 3.6 : 5.3	1 : 4.1 : 5.1	1 : 4.7 : 4.8	1 : 5.3 : 4.4
	3 " 4	1 : 2.8 : 4.8	1 : 3.2 : 4.8	1 : 3.6 : 4.6	1 : 4.0 : 4.4	1 : 4.6 : 4.0
	6 " 7	**1 : 2.1 : 4.0**	**1 : 2.5 : 4.0**	**1 : 2.8 : 3.9**	**1 : 3.2 : 3.7**	**1 : 3.5 : 3 4**
	8 " 10	1 : 1.5 : 3.0	1 : 1.7 : 3.0	1 : 1.9 : 2.9	1 : 2.2 : 2.8	1 : 2.5 : 2.7
$\frac{3}{4}$ to 2 in...........	$\frac{1}{2}$ to 1	1 : 3.2 : 6.2	1 : 3.6 : 6.1	1 : 4.0 : 6.0	1 : 4.6 : 5.8	1 : 5.2 : 5.4
	3 " 4	1 : 2.8 : 5.5	1 : 3.1 : 5.5	1 : 3.5 : 5.4	1 : 3.9 : 5.2	1 : 4.5 : 4.9
	6 " 7	**1 : 2.1 : 4.5**	**1 : 2.4 : 4.6**	**1 : 2.7 : 4.5**	**1 : 3.1 : 4.4**	**1 : 3.5 : 4.1**
	8 " 10	1 : 1.4 : 3.4	1 : 1.6 : 3.4	1 : 1.8 : 3.4	1 : 2.1 : 3.4	1 : 2.4 : 3.3
$\frac{3}{4}$ to 3 in...........	$\frac{1}{2}$ to 1	1 : 3.2 : 7.1	1 : 3.6 : 7.1	1 : 4.0 : 7.0	1 : 4.6 : 6.9	1 : 5.2 : 6.6
	3 " 4	1 : 2.7 : 6.3	1 : 3.0 : 6.3	1 : 3.4 : 6.3	1 : 4.0 : 6.2	1 : 4.5 : 5.9
	6 " 7	**1 : 2.1 : 5.1**	**1 : 2.4 : 5.2**	**1 : 2.7 : 5.2**	**1 : 3.1 : 6.1**	**1 : 3.5 : 4.9**
	8 " 10	1 : 1.4 : 3.8	1 : 1.6 : 3.9	1 : 1.8 : 3.9	1 : 2.1 : 3.9	1 : 2.4 : 3.8

PROPORTIONS FOR 2000 LB. PER SQ. IN. CONCRETE.

Proportions are expressed by volume as follows: Portland Cement : Fine Aggregate : Coarse Aggregate.

Thus 1 : 2.6 : 4.6 indicates 1 part by volume of portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate.

Size of Coarse Aggregate.	Slump, in.	Size of Fine Aggregate.				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – $\frac{3}{8}$ in.
None	$\frac{1}{2}$ to 1	1 : 2.2	1 : 2.6	1 : 3.0	1 : 3.5	1 : 4.1
	3 " 4	1 : 1.9	1 : 2.2	1 : 2.6	1 : 3.0	1 : 3.5
	6 " 7	1 : 1.5	1 : 1.7	1 : 2.0	1 : 2.3	1 : 2.7
	8 " 10	1 : 1.0	1 : 1.1	1 : 1.3	1 : 1.6	1 : 1.8
No. 4 to $\frac{3}{4}$ in.	$\frac{1}{2}$ to 1	1 : 2.1 : 3.8	1 : 2.3 : 3.7	1 : 2.6 : 3.5	1 : 3.0 : 3.1	1 : 3.6 : 2.8
	3 " 4	1 : 1.7 : 3.3	1 : 1.9 : 3.2	1 : 2.2 : 3.1	1 : 2.6 : 2.8	1 : 3.0 : 2.4
	6 " 7	1 : 1.3 : 2.7	1 : 1.4 : 2.6	1 : 1.7 : 2.5	1 : 1.9 : 2.3	1 : 2.3 : 2.1
	8 " 10	1 : 0.8 : 1.9	1 : 0.9 : 1.9	1 : 1.0 : 1.8	1 : 1.2 : 1.7	1 : 1.5 : 1.6
No. 4 to 1 in.	$\frac{1}{2}$ to 1	1 : 1.9 : 4.5	1 : 2.2 : 4.3	1 : 2.5 : 4.2	1 : 2.8 : 3.9	1 : 3.4 : 3.6
	3 " 4	1 : 1.6 : 3.9	1 : 1.8 : 3.8	1 : 2.1 : 3.7	1 : 2.4 : 3.5	1 : 2.8 : 3.2
	6 " 7	1 : 1.2 : 3.1	1 : 1.3 : 3.1	1 : 1.5 : 3.0	1 : 1.8 : 2.9	1 : 2.1 : 2.7
	8 " 10	1 : 0.7 : 2.2	1 : 0.8 : 2.2	1 : 1.0 : 2.3	1 : 1.1 : 2.1	1 : 1.3 : 2.0
No. 4 to $1\frac{1}{2}$ in.	$\frac{1}{2}$ to 1	1 : 1.9 : 5.0	1 : 2.1 : 4.9	1 : 2.4 : 4.9	1 : 2.7 : 4.6	1 : 3.2 : 4.4
	3 " 4	1 : 1.6 : 4.4	1 : 1.7 : 4.3	1 : 2.0 : 4.2	1 : 2.4 : 4.0	1 : 2.7 : 3.8
	6 " 7	1 : 1.1 : 3.5	1 : 1.3 : 3.5	1 : 1.4 : 3.5	1 : 1.7 : 3.4	1 : 2.0 : 3.2
	8 " 10	1 : 0.7 : 2.5	1 : 0.8 : 2.5	1 : 0.9 : 2.5	1 : 1.0 : 2.4	1 : 1.2 : 2.3
No. 4 to 2 in.	$\frac{1}{2}$ to 1	1 : 1.7 : 5.8	1 : 1.9 : 5.7	1 : 2.1 : 5.8	1 : 2.4 : 5.6	1 : 2.8 : 5.5
	3 " 4	1 : 1.4 : 5.0	1 : 1.5 : 5.0	1 : 1.8 : 5.0	1 : 2.0 : 4.9	1 : 2.3 : 4.7
	6 " 7	1 : 1.0 : 4.1	1 : 1.1 : 4.1	1 : 1.2 : 4.1	1 : 1.4 : 4.1	1 : 1.7 : 3.9
	8 " 10	1 : 0.6 : 2.9	1 : 0.7 : 2.9	1 : 0.7 : 3.0	1 : 0.8 : 2.9	1 : 1.0 : 2.9
$\frac{3}{8}$ to 1 in.	$\frac{1}{2}$ to 1	1 : 2.2 : 4.4	1 : 2.5 : 4.2	1 : 2.8 : 4.1	1 : 3.3 : 3.8	1 : 3.8 : 3.4
	3 " 4	1 : 1.9 : 3.8	1 : 2.1 : 3.7	1 : 2.4 : 3.6	1 : 2.8 : 3.4	1 : 3.2 : 3.1
	6 " 7	1 : 1.4 : 3.1	1 : 1.5 : 3.0	1 : 1.8 : 3.0	1 : 2.1 : 2.8	1 : 2.4 : 2.5
	8 " 10	1 : 0.9 : 2.2	1 : 1.0 : 2.2	1 : 1.1 : 2.2	1 : 1.3 : 2.0	1 : 1.5 : 1.9
$\frac{3}{8}$ to $1\frac{1}{2}$ in.	$\frac{1}{2}$ to 1	1 : 2.2 : 4.9	1 : 2.5 : 4.8	1 : 2.8 : 4.7	1 : 3.2 : 4.6	1 : 3.7 : 4.2
	3 " 4	1 : 1.9 : 4.3	1 : 2.1 : 4.2	1 : 2.4 : 4.1	1 : 2.7 : 4.0	1 : 3.1 : 3.7
	6 " 7	1 : 1.4 : 3.5	1 : 1.5 : 3.4	1 : 1.7 : 3.4	1 : 2.0 : 3.3	1 : 2.3 : 3.1
	8 " 10	1 : 0.9 : 2.5	1 : 1.0 : 2.5	1 : 1.1 : 2.4	1 : 1.3 : 2.4	1 : 1.5 : 2.3
$\frac{3}{8}$ to 2 in.	$\frac{1}{2}$ to 1	1 : 2.1 : 5.6	1 : 2.3 : 5.5	1 : 2.6 : 5.5	1 : 3.0 : 5.4	1 : 3.5 : 5.1
	3 " 4	1 : 1.7 : 4.8	1 : 2.0 : 4.8	1 : 2.2 : 4.8	1 : 2.5 : 4.7	1 : 2.9 : 4.4
	6 " 7	1 : 1.3 : 4.0	1 : 1.4 : 3.9	1 : 1.6 : 3.9	1 : 1.8 : 3.9	1 : 2.1 : 3.8
	8 " 10	1 : 0.8 : 2.9	1 : 0.9 : 2.9	1 : 1.0 : 2.9	1 : 1.2 : 2.9	1 : 1.3 : 2.8
$\frac{3}{4}$ to $1\frac{1}{2}$ in.	$\frac{1}{2}$ to 1	1 : 2.6 : 4.5	1 : 2.9 : 4.5	1 : 3.3 : 4.4	1 : 3.8 : 4.2	1 : 4.3 : 3.9
	3 " 4	1 : 2.2 : 3.9	1 : 2.5 : 3.9	1 : 2.8 : 3.8	1 : 3.2 : 3.6	1 : 3.6 : 3.3
	6 " 7	1 : 1.6 : 3.2	1 : 1.8 : 3.2	1 : 2.1 : 3.1	1 : 2.4 : 3.0	1 : 2.7 : 2.8
	8 " 10	1 : 1.0 : 2.3	1 : 1.2 : 2.3	1 : 1.4 : 2.2	1 : 1.6 : 2.2	1 : 1.8 : 2.1
$\frac{3}{4}$ to 2 in.	$\frac{1}{2}$ to 1	1 : 2.5 : 5.2	1 : 2.8 : 5.2	1 : 3.2 : 5.1	1 : 3.6 : 5.0	1 : 4.1 : 4.7
	3 " 4	1 : 2.1 : 4.5	1 : 2.4 : 4.5	1 : 2.7 : 4.4	1 : 3.1 : 4.3	1 : 3.5 : 4.0
	6 " 7	1 : 1.6 : 3.7	1 : 1.8 : 3.7	1 : 2.0 : 3.7	1 : 2.3 : 3.6	1 : 2.6 : 3.5
	8 " 10	1 : 1.0 : 2.6	1 : 1.1 : 2.7	1 : 1.3 : 2.6	1 : 1.5 : 2.7	1 : 1.7 : 2.6
$\frac{3}{4}$ to 3 in.	$\frac{1}{2}$ to 1	1 : 2.5 : 6.0	1 : 2.9 : 5.9	1 : 3.2 : 5.9	1 : 3.6 : 5.8	1 : 4.1 : 5.6
	3 " 4	1 : 2.1 : 5.1	1 : 2.4 : 5.2	1 : 2.7 : 5.2	1 : 3.1 : 5.1	1 : 3.5 : 4.9
	6 " 7	1 : 1.5 : 4.1	1 : 1.7 : 4.2	1 : 2.0 : 4.2	1 : 2.3 : 4.2	1 : 2.5 : 4.0
	8 " 10	1 : 1.0 : 2.9	1 : 1.1 : 3.0	1 : 1.3 : 3.0	1 : 1.5 : 3.0	1 : 1.7 : 3.0

PROPORTIONS FOR 2500 LB. PER SQ. IN. CONCRETE.

Proportions are expressed by volume as follows: Portland Cement : Fine Aggregate : Coarse Aggregate.
Thus 1 : 2.6 : 4.6 indicates 1 part by volume of portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate

Size of Coarse Aggregate.	Slump, in.	Size of Fine Aggregate.				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – ⅜ in.
None	½ to 1	1 : 1.8	1 : 2.1	1 : 2.4	1 : 2.9	1 : 3.3
	3 " 4	1 : 1.5	1 : 1.8	1 : 2.1	1 : 2.4	1 : 2.8
	6 " 7	1 : 1.1	1 : 1.3	1 : 1.6	1 : 1.8	1 : 2.1
	8 " 10	1 : 0.7	1 : 0.8	1 : 0.9	1 : 1.1	1 : 1.3
No. 4 to ¾ in.	½ to 1	1 : 1.6 : 3.2	1 : 1.8 : 3.1	1 : 2.1 : 3.0	1 : 2.4 : 2.7	1 : 2.9 : 2.4
	3 " 4	1 : 1.3 : 2.8	1 : 1.5 : 2.7	1 : 1.7 : 2.6	1 : 2.0 : 2.4	1 : 2.4 : 2.2
	6 " 7	1 : 1.0 : 2.2	1 : 1.1 : 2.2	1 : 1.3 : 2.1	1 : 1.5 : 2.0	1 : 1.8 : 1.8
	8 " 10	1 : 0.5 : 1.4	1 : 0.6 : 1.4	1 : 0.7 : 1.4	1 : 0.8 : 1.4	1 : 1.0 : 1.3
No. 4 to 1 in.	½ to 1	1 : 1.5 : 3.7	1 : 1.7 : 3.7	1 : 2.0 : 3.5	1 : 2.2 : 3.4	1 : 2.7 : 3.1
	3 " 4	1 : 1.2 : 3.3	1 : 1.4 : 3.2	1 : 1.6 : 3.1	1 : 1.9 : 3.0	1 : 2.2 : 2.7
	6 " 7	1 : 0.9 : 2.6	1 : 1.0 : 2.5	1 : 1.1 : 2.5	1 : 1.3 : 2.4	1 : 1.6 : 2.3
	8 " 10	1 : 0.5 : 1.7	1 : 0.6 : 1.7	1 : 0.6 : 1.7	1 : 0.7 : 1.6	1 : 0.9 : 1.5
No. 4 to 1½ in.	½ to 1	1 : 1.4 : 4.2	1 : 1.6 : 4.1	1 : 1.9 : 4.1	1 : 2.2 : 4.0	1 : 2.5 : 3.8
	3 " 4	1 : 1.2 : 3.7	1 : 1.3 : 3.6	1 : 1.5 : 3.6	1 : 1.8 : 3.5	1 : 2.1 : 3.3
	6 " 7	1 : 0.9 : 2.9	1 : 0.9 : 2.8	1 : 1.1 : 2.8	1 : 1.3 : 2.8	1 : 1.5 : 2.6
	8 " 10	1 : 0.5 : 1.9	1 : 0.5 : 1.9	1 : 0.6 : 1.9	1 : 0.7 : 1.8	1 : 0.8 : 1.8
No. 4 to 2 in.	½ to 1	1 : 1.3 : 4.9	1 : 1.4 : 4.8	1 : 1.6 : 4.9	1 : 1.9 : 4.8	1 : 2.2 : 4.7
	3 " 4	1 : 1.1 : 4.3	1 : 1.2 : 4.2	1 : 1.3 : 4.3	1 : 1.6 : 4.2	1 : 1.8 : 4.1
	6 " 7	1 : 0.7 : 3.3	1 : 0.8 : 3.3	1 : 0.9 : 3.4	1 1.1 : 3.3	1 : 1.2 : 3.3
	8 " 10	1 : 0.4 : 2.2	1 : 2.4 : 2.2	1 : 0.5 : 2.2	1 : 0.6 : 2.2	1 : 0.6 : 2.2
⅜ to 1 in.	½ to 1	1 : 1.8 : 3.7	1 : 2.0 : 3.6	1 : 2.3 : 3.5	1 : 2.6 : 3.3	1 : 3.0 : 2.9
	3 " 4	1 : 1.4 : 3.2	1 : 1.6 : 3.1	1 : 1.9 : 2.9	1 : 2.2 : 2.9	1 : 2.5 : 2.6
	6 " 7	1 : 1.0 : 2.5	1 : 1.2 : 2.5	1 : 1.3 : 2.4	1 : 1.6 : 2.3	1 : 1.8 : 2.2
	8 " 10	1 : 0.6 : 1.6	1 : 0.7 : 1.6	1 : 0.8 : 1.6	1 : 0.9 : 1.6	1 : 1.0 : 1.5
⅜ to 1½ in.	½ to 1	1 : 1.7 : 4.1	1 : 1.9 : 4.1	1 : 2.2 : 4.0	1 : 2.5 : 3.9	1 : 2.9 : 3.6
	3 " 4	1 : 1.5 : 3.6	1 : 1.6 : 3.6	1 : 1.8 : 3.5	1 : 2.1 : 3.4	1 : 2.3 : 3.2
	6 " 7	1 : 1.0 : 2.9	1 : 1.2 : 2.8	1 : 1.3 : 2.8	1 : 1.5 : 2.7	1 : 1.8 : 2.6
	8 " 10	1 : 0.6 : 1.9	1 : 0.6 : 1.9	1 : 0.8 : 1.8	1 : 0.9 : 1.8	1 : 1.0 : 1.8
⅜ to 2 in.	½ to 1	1 : 1.7 : 4.7	1 : 1.8 : 4.7	1 : 2.1 : 4.7	1 : 2.4 : 4.6	1 : 2.7 : 4.4
	3 " 4	1 : 1.4 : 4.1	1 : 1.5 : 4.1	1 : 1.7 : 4.1	1 : 2.0 : 4.0	1 : 2.3 : 3.9
	6 " 7	1 : 1.0 : 3.2	1 : 1.1 : 3.2	1 : 1.2 : 3.2	1 : 1.4 : 3.2	1 : 1.6 : 3.1
	8 " 10	1 : 0.5 : 2.1	1 : 0.6 : 2.1	1 : 0.7 : 2.2	1 : 0.8 : 2.2	1 : 0.9 : 2.1
¾ to 1½ in.	½ to 1	1 : 2.0 : 3.8	1 : 2.3 : 3.8	1 : 2.6 : 3.7	1 : 3.0 : 3.6	1 : 3.4 : 3.3
	3 " 4	1 : 1.7 : 3.3	1 : 2.0 : 3.3	1 : 2.2 : 3.2	1 : 2.5 : 3.2	1 : 2.9 : 2.9
	6 " 7	1 : 1.2 : 2.6	1 : 1.4 : 2.6	1 : 1.6 : 2.6	1 : 1.9 : 2.5	1 : 2.1 : 2.3
	8 " 10	1 : 0.7 : 1.7	1 : 0.8 : 1.7	1 : 0.9 : 1.7	1 : 1.1 : 1.7	1 : 1.2 : 1.6
¾ to 2 in.	½ to 1	1 : 2.0 : 4.4	1 : 2.2 : 4.4	1 : 2.5 : 4.3	1 : 2.9 : 4.3	1 : 3.3 : 4.1
	3 " 4	1 : 1.7 : 3.8	1 : 1.9 : 3.8	1 : 2.1 : 3.8	1 : 2.5 : 3.7	1 : 2.8 : 3.6
	6 " 7	1 : 1.2 : 3.0	1 : 1.4 : 3.0	1 : 1.5 : 3.0	1 : 1.8 : 3.0	1 : 2.0 : 2.8
	8 " 10	1 : 0.7 : 2.0	1 : 0.8 : 2.0	1 : 0.9 : 2.0	1 : 1.0 : 2.0	1 : 1.2 : 2.0
¾ to 3 in.	½ to 1	1 : 2.0 : 5.0	1 : 2.2 : 5.0	1 : 2.5 : 5.0	1 : 2.7 : 5.0	1 : 3.2 : 4.7
	3 " 4	1 : 1.7 : 4.3	1 : 1.9 : 4.3	1 : 2.1 : 4.3	1 : 2.4 : 4.3	1 : 2.7 : 4.1
	6 " 7	1 : 1.2 : 3.3	1 : 1.4 : 3.4	1 : 1.5 : 3.4	1 : 1.8 : 3.4	1 : 2.0 : 3.3
	8 " 10	1 : 0.7 : 2.2	1 : 0.8 : 2.2	1 : 0.9 : 2.2	1 : 1.0 : 2.3	1 : 1.2 : 2.3

PROPORTIONS FOR 3000 LB. PER SQ. IN. CONCRETE.

Proportions are expressed by volume as follows: Portland Cement : Fine Aggregate : Coarse Aggregate.

Thus 1 : 2.6 : 4.6 indicates 1 part by volume of portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate.

Size of Coarse Aggregate.	Slump, in.	Size of Fine Aggregate.				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – $\frac{3}{8}$ in.
None	$\frac{1}{2}$ to 1	1 : 1.5	1 : 1.7	1 : 2.0	1 : 2.3	1 : 2.7
	3 " 4	1 : 1.2	1 : 1.4	1 : 1.7	1 : 1.9	1 : 2.3
	6 " 7	1 : 0.9	1 : 1.0	1 : 1.2	1 : 1.4	1 : 1.6
	8 " 10	1 : 0.5	1 : 0.6	1 : 0.7	1 : 0.8	1 : 0.9
No. 4 to $\frac{3}{4}$ in.	$\frac{1}{2}$ to 1	1 : 1.3 : 2.7	1 : 1.5 : 2.6	1 : 1.7 : 2.5	1 : 1.9 : 2.4	1 : 2.3 : 2.1
	3 " 4	1 : 1.0 : 2.3	1 : 1.2 : 2.2	1 : 1.4 : 2.2	1 : 1.6 : 2.0	1 : 1.9 : 1.8
	6 " 7	1 : 0.7 : 1.7	1 : 0.8 : 1.7	1 : 0.9 : 1.7	1 : 1.1 : 1.6	1 : 1.3 : 1.4
	8 " 10	1 : 0.3 : 1.0	1 : 0.4 : 1.0	1 : 0.5 : 1.0	1 : 0.5 : 1.0	1 : 0.6 : 0.9
No. 4 to 1 in.	$\frac{1}{2}$ to 1	1 : 1.2 : 3.1	1 : 1.3 : 3.1	1 : 1.5 : 3.0	1 : 1.8 : 2.9	1 : 2.1 : 2.7
	3 " 4	1 : 0.9 : 2.7	1 : 1.1 : 2.6	1 : 1.2 : 2.6	1 : 1.4 : 2.5	1 : 1.7 : 2.3
	6 " 7	1 : 0.6 : 2.0	1 : 0.7 : 2.0	1 : 0.8 : 2.0	1 : 0.9 : 1.9	1 : 1.1 : 1.8
	8 " 10	1 : 0.3 : 1.2	1 : 0.3 : 1.2	1 : 0.4 : 1.2	1 : 0.5 : 1.2	1 : 0.6 : 1.2
No. 4 to $1\frac{1}{2}$ in.	$\frac{1}{2}$ to 1	1 : 1.1 : 3.6	1 : 1.2 : 3.5	1 : 1.5 : 3.5	1 : 1.7 : 3.4	1 : 2.0 : 3.2
	3 " 4	1 : 0.9 : 3.0	1 : 1.0 : 2.9	1 : 1.2 : 2.9	1 : 1.4 : 2.9	1 : 1.6 : 2.7
	6 " 7	1 : 0.6 : 2.2	1 : 0.7 : 2.2	1 : 0.8 : 2.2	1 : 0.9 : 2.2	1 : 1.1 : 2.1
	8 " 10	1 : 0.3 : 1.4	1 : 0.3 : 1.3	1 : 0.4 : 1.4	1 : 0.5 : 1.4	1 : 0.5 : 1.3
No. 4 to 2 in.	$\frac{1}{2}$ to 1	1 : 1.0 : 4.1	1 : 1.1 : 4.1	1 : 1.2 : 4.1	1 : 1.4 : 4.1	1 : 1.6 : 4.0
	3 " 4	1 : 0.8 : 3.4	1 : 0.9 : 3.4	1 : 1.0 : 3.5	1 : 1.1 : 3.4	1 : 1.3 : 3.4
	6 " 7	1 : 0.5 : 2.6	1 : 0.6 : 2.6	1 : 0.6 : 2.7	1 : 0.7 : 2.6	1 : 0.9 : 2.6
	8 " 10	1 : 0.2 : 1.6	1 : 0.3 : 1.6	1 : 0.3 : 1.7	1 : 0.4 : 1.7	1 : 0.4 : 1.7
$\frac{3}{8}$ to 1 in.	$\frac{1}{2}$ to 1	1 : 1.4 : 3.1	1 : 1.5 : 3.0	1 : 1.8 : 2.9	1 : 2.1 : 2.8	1 : 2.4 : 2.6
	3 " 4	1 : 1.1 : 2.6	1 : 1.3 : 2.6	1 : 1.5 : 2.5	1 : 1.7 : 2.4	1 : 2.0 : 2.2
	6 " 7	1 : 0.8 : 2.0	1 : 0.8 : 2.0	1 : 1.0 : 1.9	1 : 1.1 : 1.9	1 : 1.3 : 1.8
	8 " 10	1 : 0.4 : 1.2	1 : 0.4 : 1.2	1 : 0.5 : 1.2	1 : 0.6 : 1.2	1 : 0.7 : 1.1
$\frac{3}{8}$ to $1\frac{1}{2}$ in.	$\frac{1}{2}$ to 1	1 : 1.4 : 3.5	1 : 1.5 : 3.4	1 : 1.7 : 3.4	1 : 2.0 : 3.3	1 : 2.3 : 3.1
	3 " 4	1 : 1.1 : 3.0	1 : 1.2 : 2.9	1 : 1.4 : 2.9	1 : 1.6 : 2.8	1 : 1.9 : 2.6
	6 " 7	1 : 0.6 : 2.2	1 : 0.8 : 2.2	1 : 1.0 : 2.2	1 : 1.1 : 2.1	1 : 1.3 : 2.0
	8 " 10	1 : 0.4 : 1.4	1 : 0.4 : 1.4	1 : 0.5 : 1.4	1 : 0.6 : 1.3	1 : 0.7 : 1.3
$\frac{3}{8}$ to 2 in.	$\frac{1}{2}$ to 1	1 : 1.3 : 4.0	1 : 1.4 : 4.0	1 : 1.6 : 4.0	1 : 1.9 : 3.9	1 : 2.1 : 3.8
	3 " 4	1 : 1.0 : 3.4	1 : 1.2 : 3.4	1 : 1.3 : 3.3	1 : 1.5 : 3.3	1 : 1.7 : 3.2
	6 " 7	1 : 0.7 : 2.6	1 : 0.8 : 2.5	1 : 0.9 : 2.6	1 : 1.0 : 2.6	1 : 1.1 : 2.5
	8 " 10	1 : 0.4 : 1.6	1 : 0.4 : 1.6	1 : 0.5 : 1.6	1 : 0.5 : 1.6	1 : 0.6 : 1.6
$\frac{3}{4}$ to $1\frac{1}{2}$ in.	$\frac{1}{2}$ to 1	1 : 1.6 : 3.2	1 : 1.8 : 3.2	1 : 2.1 : 3.2	1 : 2.4 : 3.1	1 : 2.7 : 2.9
	3 " 4	1 : 1.3 : 2.7	1 : 1.5 : 2.7	1 : 1.7 : 2.7	1 : 2.0 : 2.6	1 : 2.3 : 2.5
	6 " 7	1 : 0.9 : 2.0	1 : 1.0 : 2.1	1 : 1.2 : 2.0	1 : 1.4 : 2.0	1 : 1.5 : 1.8
	8 " 10	1 : 0.5 : 1.2	1 : 0.5 : 1.3	1 : 0.6 : 1.3	1 : 0.7 : 1.3	1 : 0.8 : 1.2
$\frac{3}{4}$ to 2 in.	$\frac{1}{2}$ to 1	1 : 1.6 : 3.7	1 : 1.8 : 3.7	1 : 2.0 : 3.7	1 : 2.4 : 3.6	1 : 2.6 : 3.5
	3 " 4	1 : 1.3 : 3.1	1 : 1.5 : 3.1	1 : 1.6 : 3.1	1 : 1.9 : 3.1	1 : 2.2 : 3.0
	6 " 7	1 : 0.9 : 2.4	1 : 1.1 : 2.4	1 : 1.1 : 2.4	1 : 1.3 : 2.4	1 : 1.5 : 2.3
	8 " 10	1 : 0.5 : 1.5	1 : 0.5 : 1.5	1 : 0.6 : 1.5	1 : 0.7 : 1.5	1 : 0.8 : 1.5
$\frac{3}{4}$ to 3 in.	$\frac{1}{2}$ to 1	1 : 1.6 : 4.2	1 : 1.8 : 4.2	1 : 2.0 : 4.2	1 : 2.3 : 4.1	1 : 2.6 : 4.0
	3 " 4	1 : 1.3 : 3.5	1 : 1.5 : 3.6	1 : 1.6 : 3.6	1 : 1.9 : 3.6	1 : 2.1 : 3.5
	6 " 7	1 : 0.9 : 2.6	1 : 1.0 : 2.6	1 : 1.1 : 2.6	1 : 1.3 : 2.6	1 : 1.4 : 2.6
	8 " 10	1 : 0.5 : 1.6	1 : 0.5 : 1.6	1 : 0.6 : 1.7	1 : 0.7 : 1.7	1 : 0.8 : 1.7

APPENDIX 17.

EFFECT OF OILS AND MISCELLANEOUS LIQUIDS ON CONCRETE AND METHOD OF PROTECTIVE TREATMENT WHERE REQUIRED

Liquid	Effect on Untreated Concrete	Surface Treatment
	MINERAL OILS[1]	
30° Baumé or heavier....	Good concrete unaffected. Very slight surface penetration.	None—Good concrete well spaded or cement mortar finish sufficient.
Fuel Oils above 30° Baumé. Distillates. Gas and Lubricating Oils.[2]..............	Good concrete unaffected. More penetration than for heavy oils.	Coatings of the magnesium fluosilicate class, glues or varnishes required for storage tanks.
Kerosene, Gasoline, Benzine.............	Good concrete unaffected. Considerable penetration.	Gasoline-proof coatings producing glazed surface or treatment with iron compounds.
	ANIMAL OILS (SOLID FATS)[3]	
Lard and Lard Oil......	May attack concrete slowly, particularly if in melted condition.	Various proprietary compounds recommended by manufacturers of technical paints.
Goose Fat, Beef Marrow, Beef and Mutton Tallow and Tallow Oil.................	No definite information. Probably similar to lard oil.	Probably similar to that for lard oil.
	ANIMAL OILS (LIQUID FATS)	
Marine:		
Menhaden Oil.........	No effect on good concrete.	Cushman's tests indicate various coatings no better than plain concrete.
Cod Liver Oil....... Shark Liver Oil. .... Seal and Whale Oil..	More or less disintegration depending on quality of concrete.	Various proprietary compounds recommended by manufacturers of technical paints.
Terrestrial:		
Sheep's foot...........	"	"
Horse's foot...........	"	"
Neat's foot.[4]..........	No effect on good concrete.	No treatment required.

[1] Signal oil, used by railroads, is a mixture of animal fat with mineral oil. Probably has about the same effect on concrete as lard oils.

[2] Some lubricating oils are mixtures of mineral and animal oils.

[3] Bureau of Standards tests with concrete tanks show slight roughening of surface at end of 12 months and considerable deposit on surface through saponification.

[4] Bureau of Standards noted slight deposit due to saponification at end of 12 months.

APPENDIX 17—*Continued.*

Liquid	Effect on Untreated Concrete	Surface Treatment
VEGETABLE OILS (SOLID FATS)		
Cocoanut Oil.[1]	Some action if stored in closed tank. Progressive disintegration if in contact with surfaces exposed to air.	Several proprietary compounds seem to have proved effective on floors. Sodium silicate or magnesium fluosilicate treatment apparently sufficient for closed tanks.
Palm Oil	No information.	No information.
VEGETABLE OILS		
Drying: Hemp Oil Poppyseed Oil Tobacco Oil	No information.	No information.
Linseed Oil.[2] Rosin Oil Turpentine	No effect on good concrete. Considerable penetration of turpentine.	Cushman's tests indicate various coatings no better than plain concrete for linseed and rosin oils.
Semi-Drying: Cottonseed Oil	No action if stored in closed tank of good concrete.	Same as for cocoanut oil.
Rape Seed Oil Castor Oil Mustard Oil	Progressive disintegration if in contact with surfaces exposed to air.	
Non-Drying: Olive Oil	Probably some action.	Cushman's tests show proprietary coatings of varnish type effective.
Butter Oil. (?)		

[1] Bureau of Standards tests with concrete tanks show considerable softening and roughening of surface at end of 12 months.

[2] Bureau of Standards noted at end of 12 months considerable deposit on surfaces of concrete tanks containing both boiled and raw linseed oil, due to saponification, but concrete showed no deterioration.

APPENDIX 17—*Continued.*

Liquid	Effect on Untreated Concrete	Surface Treatment
MISCELLANEOUS LIQUIDS		
Tanning Liquors.	Acid liquors show considerable effect. Other tanning extracts have no action.	Bituminous acid-proof paints effective for tanks holding acid tanning solutions. Good concrete with or without mortar finish sufficient for other tanning liquors.
Sulfite Liquor.	Attacks untreated concrete tanks.	Cushman's tests indicate bituminous acid-proof paints effective. Paraffin coating fair.
Cider Vinegar.	Acetic acid attacks concrete.	Cushman's tests indicate bituminous acid-proof paints effective. Paraffin coatings applied hot also useful for tanks.
Sauerkraut Brine.	No action on good concrete.	Cushman's tests show special treatments no better than untreated concrete.
Whey.	More or less action depending on quality of concrete.	Cushman's tests show proprietary coating of varnish type effective. Sodium silicate treatment used on storage tanks.
Buttermilk.	No action on good concrete.	Untreated tanks used successfully to store buttermilk.
Molasses.	No action in closed concrete.	Good concrete well spaded or finished with cement mortar sufficient. Annapolis mixture sometimes used.
Sulfuric Acid Solutions. . .	Progressive disintegration, particularly where concrete is subject to abrasion.	Bituminous acid-proof paints or mastic coating effective.

APPENDIX 18.

SUPERVISION AND INSPECTION OF CONCRETE CONSTRUCTION.

1. All work shall be under the supervision of the Engineer, under whose direction the work is to be conducted in accordance with the provisions of the specifications; the Engineer shall interpret the meaning and intent of the drawings and specifications, and pass upon the material and workmanship, and his acceptance shall be a condition precedent to payment. In case of disputes, or of unforeseen conditions the decision of the Engineer as to quantities, quality and acceptability of the work shall be binding.

2. Line and grade will be given by the Engineer as provided under the contract. The Contractor shall inform the Engineer a reasonable time in advance so that lines and grades may be conveniently and accurately placed. Such points or marks as may be given by the Engineer shall be carefully preserved by the Contractor.

3. The Inspector as a representative of the Engineer shall be in immediate charge of the inspection of material and of placing material in the finished work. The Contractor shall at all times give to the Owner, the Engineer or Inspector, entrance to the work and to the place of manufacture of any materials entering into the work, and facilities for inspecting the work or materials, both at completion and in process of manufacture and construction. The manufacturer shall furnish the Engineer such chemical and physical records of the materials as may be required under the contract and any other records pertaining to the quality of the material that the manufacturer may possess shall be available for the information of the Engineer or Inspector.

4. The Inspector shall be present before starting any unit of field work and during the process of construction of any unit of the structure, to see that the work is done according to the provisions of the specifications, as interpreted by the Engineer.

5. The tests required under these specifications will be made by the Engineer and unless otherwise specifically provided for will be carried out at the expense of the Owner. The Contractor shall afford every opportunity for the proper conduct of the tests and he shall further provide such facilities for obtaining, handling, storing, and

testing specimens and samples as the Engineer may require. For such work the Contractor shall receive compensation for actual expense for labor and materials.

6. The inspection of the work shall not relieve the Contractor of his obligations under the contract. Defective work shall be made good and unsuitable materials rejected, even though such work and materials have previously been accepted.

Mr. Godfrey. EDWARD GODFREY *(By Letter)*.—C. A. P. Turner, of Minneapolis, Minn., sent me a copy of a criticism of the Joint Committee Report by a committee of the Engineers' Club of Minneapolis. Presumably the Joint Committee Report is to be up for discussion, or at least this criticism will be read at the convention. In either event, responding to Mr. Turner's suggestion I should like to present before the convention some remarks on the subject.

Members of the Concrete Institute know my attitude in regard to three principal features of design in reinforced concrete. One is the rodded column, which, as this criticism from Minneapolis (hereinafter referred to as the "criticism") points out, has been present in a very uncomfortable number of wrecks, too uncomfortable for any advocate of this type of column to talk about. The second is the short shear member, the only type of shear reinforcement recognized in any standard work, and the type which I have unequivocally condemned for nearly two decades. Countless beams that have broken away from their supports have demonstrated clearly the inadequacy of alleged shear reinforcement that does not reach the support nor cross the sections which break in these failures. The third is the system of coefficients for flat slabs.

The "criticism" is eminently correct in its statement that building codes should specify a standard grade of cement, sand and aggregate with standard mixtures. For many years steel structures were built on dual specifications as to the grade of steel, medium steel and soft steel being the two grades specified, sometimes with different unit stresses for the two grades. Manufacturers regularly made steel that would meet the requirements of either grade (in the overlap), and engineers imagined that they had a higher or softer grade of steel, when the only difference was the word that happened to be written on the order. Considering conditions on a job the fancy stuff on concrete mixes in the Joint Committee Report duplicates a worse absurdity than the steel specifications many fold.

When one looks at it squarely, it is absurd to come up to the point of pouring concrete for a structure not knowing what unit stress the concrete is going to be capable of sustaining, for that is what is implied by the Joint Committee specifications. The unit stress is dependent upon the compressive stress that can be wrung out of the materials. One bidder, with better artists in his laboratory, may obtain much higher compressive test results than another.

Discussion as to trapezoidal and rhombic shear distortion in beams has little meaning, when the actual beam can fail in actual shear close to the support, when the actual shear is the greatest, and never even stress the alleged shear reinforcement, which in all standard designs is located elsewhere than across this section of maximum shear. Hundreds of beams have failed in this way in buildings that have collapsed. The thing that is important is to have shear reinforcement that would have to sever before

the beam that it reinforces can fail. This no standard, except the alternate introduced into the Joint Committee Report, accomplishes. Mr. Godfrey.

The "criticism," while it finds fault, and justly, with the rodded column of the Joint Committee Report, does not go far enough in its substitution to insure a tough and safe column.

A tabular comparison between values of rodded and spiral columns, just received from Walter H. Wheeler, of Minneapolis, shows astounding data. The comparisons of this table are enough to brand the Joint Committee Report as a dangerous guide in the design of columns. That the rodded column, which has been the chief feature of the design of all the long list of large wrecks, should be exalted above the hooped column in the manner shown in this table is absolutely inexcusable.

The "criticism" refers to tests where concrete expanded on setting. These test results contradict a vast amount of practical work where open cracks in long walls and pavements indicate clearly shrinkage of concrete on setting.

As to the flat slab. As the present writer is doubtless the former member of the Joint Committee referred to who maintains that tension in concrete accounts for the good showing in measured steel stress, it is fitting that the unanswerable arguments of fact should be restated. No one can deny that the flat slab that is loaded has deflection; and that deflection is impossible without tension in the concrete; and that, if there is tension in the concrete, that tension must be helping to resist the moments; and that a steel bar embedded in concrete is subject to one of two conditions—either it is resisting only a portion of the tension or else, if a crack occurs, the stress cannot be measured, because the measured length is not subject to the maximum stress—only the portion at the crack. The last statement proves the inherent error of all stresses in embedded steel measured by strain gages.

Because a flat slab may fail, if not reinforced, under a much smaller load than if reinforced, does not vitiate the argument that tension in the concrete helps to carry the load and, in fact, carries a large part of it. Reinforcement in many directions prevents the start and spread of cracks. In a plain slab a weak point or shrinkage may start a crack, and because of the brittleness of the concrete, this crack may spread and cause failure. I have myself tested a floor where slabs on a 4-ft. span with a thickness of only about 2½ in. carried without distress 250 lb. per square foot. There was not a wire of reinforcement in the slabs and many of the panels were cracked down the middle on account of shrinkage of the concrete. What was carrying this load, if it was not tension in the concrete?

The experiment in the "criticism" of a slab supported on two edges and on four edges and loaded at the center does not have application to the Joint Committee's rules for such flat plates, as a uniform load and not a center load is referred to.

A. V. BEKAY *(By Letter)*.—The following criticism is made of the Joint Committee's report: Mr. Bekay.

Mr. Bekay. PAR. 145.—*Thickness of Flat Slab.*—The formula No. 37 is too complicated. A much better one is that used by the New York City building code; viz., $t = 0.02 \sqrt{w + 1}$.

PAR. 148.—According to this paragraph if the marginal beam is of a depth less or equal to the drop panel, then it need not be designed to take any part of the slab load in addition to the load superimposed directly upon it. But just as soon as the marginal beam is made deeper than the dropped panel, then it has to be designed to take at least one-fourth of the slab load that the adjacent panels are designed for.

It does not seem reasonable to design the marginal beam for no floor load at all in the first case, and then design it for one-fourth the floor load in the second case, with no provision for a gradual increase from the first case to the second.

PAR. 153.—Too many different lengths are called for and it is far better to put in a few extra bars than so many different lengths from the practical point of erection and construction.

By adopting the proposed standard specifications, the following items would appreciably increase the cost of the structures:

1. Longer slab bars to handle.
2. Three different lengths of bars to every main band.
3. Two different lengths of bars to every mid band.
4. Design of structure would take about 20 per cent more time owing to the complicated formulas.
5. Approximately 35 per cent more steel required, than under the present New York City code.
6. The steel due to variation will cost at least $5 per ton more to place.

PAR. 165.—Permissible compressive stress in concrete limited to 400 lb. for 2,000-lb. concrete is too low. New York is using 500 lb. and this should not be decreased.

PAR. 167.—The current practice of using 50 per cent excess stresses for combined axial load and bending is not unsafe and has been the current practice in New York City for several years. There is no occasion for changing this unless it can be proved that this is unsafe.

PAR. 182.—It is inconsistent for the dowels to be extended in the pedestal to develop the full volume in tension unless these bars are actually in tension, and you cannot transmit any more stress than these bars are subject to in compression. This is simply increasing the cost of concrete work without any benefit to anyone.

Mr. Ahlers. JOHN G. AHLERS *(By Letter)*.—The following criticism is made of the Joint Committee's report in the sections given below:

41. *Protection.*—It is impractical to protect columns and the underside of slabs for seven days after being concreted and keep them moist.

45. *Rubble Concrete.*—If the rubbles are clean there will be as much chemical bond and setting on the individual pieces of rubble as there would be on the individual pieces of gravel aggregate, and there is no justification

in keeping the individual stones as far apart as the maximum size of coarse aggregate. It is impractical to place rubble so carefully in concrete work.

53. *Forms.*—It is impractical to have nails withdrawn from forms when once used.

68. *Fire Protection.*—This specification is prohibitive and unreasonable in its demands. Current practice of using ¾ in. in slabs and 1½ in. in beams has built millions of dollars' worth of buildings in which the loss due to spalling is so insignificant in percentage to the total cost that this demand is ridiculous in the penalty it would impose on the industry as a whole. This would increase the volume of the concrete 5 per cent or increase the steel correspondingly, which would mean an increase in cost of building of about 1 per cent on a total of $392,367,000 worth of industrial work for the year 1924, this being only 6 per cent of the total amount of building work in the United States, (Reference taken from annual survey of *Architectural Forum*), or a total of $3,923,670 penalty on the industry, whereas the actual amount of damage to concrete buildings during this same period did not aggregate $100,000.

70. *Joints.*—For practical reasons it is better to fill the columns to the top of the capital, as thus there are avoided laitance joints and seams far more harmful than any theoretical gain by pouring this capital with a slab. Certainly current practice is along the line of filling to the top of the capital and a better finished job is accomplished in this manner.

WALTER H. WHEELER *(By Letter).*—The report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete represents the results of a long study and investigation by a group of engineers who deserve the whole-hearted thanks of the profession. In many ways this report marks progress in the art of constructing reinforced concrete and it is to be hoped that it will aid in improving the quality of concrete and place the art of making concrete on a more exact and scientific basis. One great step has been taken in giving credit in the design of columns for stronger concrete which gives the engineer some latitude in design. **Mr. Wheeler.**

There are certain features of the specifications with which the writer cannot agree: notably, the specification covering standard sizes for reinforcing bars in Chap. IV, Sec. 22. It seems very fortunate that a group of steel manufacturers should meet and decide upon a certain schedule of sizes and shapes to be rolled and adopt those sizes without getting the opinion of the entire engineering profession upon such a drastic change. The writer is in full accord with the purpose of simplified practice when that practice does not result in injury to any of the users of the product or products involved. The Division of Simplified Practice of the U. S. Department of Commerce states clearly in its statement of its principles that the number of users affected or the value of their output is not the controlling factor. If any part of the industry is damaged that is sufficient cause for giving that part consideration. Simplified practice means eliminate waste, not make waste.

The scheduled sizes adopted by the Division of Simplified Practice contains one more size, namely, ¼ in. round, than the schedule set forth

Mr. Wheeler. in the Joint Committee specifications. The writer wishes to enter his objections to this schedule of sizes for the following reasons:

1. In the design of solid concrete slab construction, particularly light slabs, good practice requires that a maximum spacing center to center of main reinforcing bars will not exceed 1½ times the effective depth of the concrete slab.

2. There are a great many slabs constructed for which ⅜-in. round bars are too large to permit of this maximum spacing without using an excess amount of steel.

3. The mill extra for ¼-in. round bars over and above the base price is $1 per 100 lb. The mill extra on 5/16-in. round bars is 70¢ per 100 lb.; therefore, if ¼-in. round bars are used where 5/16-in. round bars should be used, approximately 10 per cent is added to the cost of the reinforcing required.

4. If ⅜-in. round bars are used and extra bars added to provide for maximum spacing, the weight of steel may be increased approximately 50 per cent above the weight required if 5/16-in. round bars were available.

5. Three-eighths-inch square bars or 7/16-in. round bars are another size for which there is a large use in reinforced-concrete construction. The mill extra for 7/16-in. round bars is 10¢ less per 100 lb. than for ⅜-in. square bars of approximately the same sectional area.

6. Sectional area of 1⅛-in. round is practically the same as 1-in. square. The sectional area of 1¼-in. round is practically the same as 1⅛-in. square.

7. Round bars in reinforced-concrete work, in my experience, are more satisfactory to handle, can be embedded in the concrete work with more satisfactory results, and where the same depths from the concrete surface are maintained the round bar has more fireproofing than the square bar, unless the square bar is placed cornerwise, which cannot readily be done.

Based upon the above considerations, the writer wishes to urge the committee to revise this schedule as follows:

Round Bars Diam. In.	Sq. In.
¼	.049
5/16	.077
⅜	.110
7/16	.150
½	.196
9/16	.249
⅝	.306
¾	.441
⅞	.601
1	.785
1⅛	.994
1¼	1.227
1⅜	1.485

Referring to Chapter VIII, Sec. 70: In the writer's practice he has never been able to discover any advantage in casting columns several hours ahead of casting slabs and beams for stories of ordinary height, provided the concrete does not contain an excessive amount of water, unless the columns contain a different concrete mixture from that in slabs and beams. He prefers to pour columns just ahead of pouring slabs and beams so that the concrete in the columns will have only about one-half hour to settle in advance of pouring slabs and beams. He prefers not have a construction joint in columns. **Mr. Wheeler.**

Referring to Chap. IV, Sec. 25: Why does the committee specify cold drawn wire, which covers wire for spiral reinforcement? Why not also rolled rods, structural or intermediate grade?

Referring to Chap. VIII, Sec. 74: From the writer's observation, expansion joints in buildings had better be omitted. In any building which is heated the temperature variation is small throughout the year. He has frequently observed that expansion joints in buildings are a source of trouble. Cracking in buildings due to temperature and shrinkage is usually well distributed and is very little affected by expansion joints 200 ft. apart.

Referring to Chap. XI of the report: The writer like to feel that engineers base their conclusions on facts. The first thing he would expect to find in a report which recommends such wide departures from common design practice would be an imposing array of facts in the form of test data or at least a complete list of references to all published test data. Instead we are given the conclusions of the committee unsupported by a single test or test reference.

Certainly if the committee expects their recommendations on design to be adopted by the profession as a standard it is up to them to show the authority for their conclusions.

The writer has prepared a tabulated comparison of the Joint Committee design of columns with that contained in the building ordinance of Minneapolis which is attached hereto, and which discloses some very remarkable values.

Test data with which the writer is familiar do not tend to inspire confidence in rodded and tied columns for high beaming values, and he has always been under the impression that columns reinforced with a combination of vertical steel and spiral hooping in proper proportions and properly distributed are much more reliable columns under a high unit compression stress than tied columns. The records of failures in reinforced-concrete buildings support this belief.

An examination of the table above referred to discloses that the Joint Committee recommends a unit stress based upon the core area of 1180 lb. per square inch for a 12 x 12-in. tied column and a unit stress upon the core area of a spiral hooped column with 4 per cent of vertical steel equal to 1325 lb. per square inch. If these two columns are compared on the basis of total sectional area we find the unit stress of the tied column is 525 lb. per square inch and of the column reinforced with verticals and spirals is 463 lb. per square inch, a most astonishing com-

Mr. Wheeler. parison! If credit is to be given to the fireproofing on a column is there any basis upon which it can be given so much more credit in a tied column that in a spiral column? If the fireproofing is destroyed by fire as it has been, are we to prefer a tied column with a unit compression stress of

TABLE 1.—COLUMN COMPARISONS: BASED ON CONCRETE STRENGTH OF 2200 LB. PER SQUARE INCH.

Column Design	Joint Committee Code			Minneapolis Building Ordinance Spiral Cols. 800 lb. per sq. in. on core 10,000 lb. per sq. in. on ver.? Considere Ration on Spirals		
	Total Load, lb.	Load per sq. in. Core, lb.	Load per sq. in. Total Area, lb.	Total Load, lb.	Load per sq. in. Core, lb.	Load per sq. in. Total Area, lb.
Size 12 by 12 in.......... Verticals 2 sq. in.......... Ties ¼ in. rd. 8 in. cc......	75,680	1,180	525	50,000	690	347
Size 12 by 12 in.......... Verticals 2 sq. in. 4%..... Spiral ¼ in. rd. 1½ in. cc..	66,690	1,325	463	95,000	1,670	660
Size 14 by 14 in.......... Verticals 3 sq. in.......... Ties ¼ in............	104,720	1,047	535	76,125	690	388
Size 14 by 14 in.......... Verticals 3 sq. in. 3½%... Spirals ¼ in. rd. 1½ cc.....	96,638	1,230	494	138,400	1,600	705
Size 16 by 16 in.......... Verticals 4 sq. in....... Ties ¼ in. rd. 8 in. cc.....	137,280	955	537	106,000	680	414
Size 16 by 16 in.......... Verticals 4 sq. in. 3½%... Spirals ¼ in. rd. 1¾ in. cc..	139,373	1,235	544	175,500	1,425	685
Size 18 by 18 in.......... Verticals 6 sq. in.......... Ties ¼ in. rd. 8 in. cc....	179,520	915	555	147,000	700	454
Size 18 by 18 in.......... Verticals 6 sq. in. 3.9%... Spirals ¼ in. rd. 1½ in. cc..	205,300	1,335	633	242,300	1,470	750
Size 24 by 24 in.......... Verticals 11 sq. in......... Ties ¼ in. rd. 8 in. cc.....	321,000	803	557	287,000	684	500
Size 24 by 24 in........... Verticals 11 sq. in. 3½%.. Spirals $\frac{5}{16}$ in. rd. 1¾ in. cc..	396,000	1,224	688	472,000	1,480	818
Size 30 by 30 in .... Verticals 18 sq. in......... Ties ¼ in. rd. 8 in. cc......	506,880	750	563	476,000	678	528
Size 30 by 30 in........... Verticals 18 sq. in. 3.4%.. Spirals 3.8 in. rd. 2 in. cc..	655,000	1,195	728	784,000	1,455	870

1180 lb. per square inch to a column with vertical reinforcing and spiral hooping and a unit stress of 1325 lb. per square inch?

Again referring to the tabulated comparison we find that the unit stress on the core area of tied columns according to the Joint Committee's

formula reduces from 1180 lb. per square inch on the core area for a 12 x 12-in. column to 750 lb. per square inch on the core area for a 30 x 30-in. column. The writer prefers to reverse the order and use higher unit stresses on large columns than on small columns if there is to be any difference. Again we find by an examination of the table that according to Joint Committee formulas a tied column is stronger than a column of equal size having 3½ per cent vertical steel and 0.9 per cent spiral reinforcing for all columns of less than 16 in. outside diameter. Mr. Wheeler.

It seems odd that the committee should introduce two items (P and A) into the formula for spirally hooped columns when one (A_s) the sectional area of vertical steel is the product of P and A and is known. At this point attention should be called to the fact that the unit stress on the core area of the 14-in. round core column calculated according to the Minneapolis building ordinance which is based on the Considere ratio for spirals is 1600 lb. per square inch. This is due to the fact that the spiral hooping is in excess of the "one-quarter of the vertical steel" required by the Joint Committee report ¼-in. wire spirals would have a spacing of 2¼ in., or more than the one-sixth of the core diameter allowed, and this spacing has been reduced which increases the amount of spiral. If allowance is made for the excess spiral the unit stress on the core area of the column designed according to Minneapolis code becomes 1450 lb. per square inch or substantially the same as for all other sizes considered except the 12 inch column which also has an excess of spiral when based upon Joint Committee "one-quarter of the vertical steel" requirement.

In the discussions which have appeared in connection with the preliminary reports of the Joint Committee the fact has been brought out by Mr. Godfrey and others, and tests have been referred to prove that very little strength is added to a column by vertical rods and light ties spaced far apart. Apparently the Joint Committee has either overlooked these tests or has ignored them.

The column tests made by several different authorities and referred to by F. R. McMillan, a member of the committee, in the 1921 *Proceedings* of the American Concrete Institute indicate that a reinforced-concrete column made of concrete with an ultimate strength of 2600 lb. per square inch and reinforced with 4.6 per cent of vertical steel and 1 per cent of spirals develops an ultimate strength of 5,060 lb. and this is for concrete 45 to 60 days old. It is fair to assume that this concrete had a strength at least 25 per cent. greater than at the age of 28 days. According to Joint Committee rules which would increase the spiral on this column to 1.15 per cent the allowable load is 1365 lb. per square inch or 27½ per cent of the yield point.

According to tests by Turner and published in "Concrete Steel Construction," Part I, by Eddy and Turner, columns having from 3.12 per cent to 5.17 per cent of vertical steel and from 2.01 per cent to 2.53 per cent of hooping developed an ultimate strength of 7,350 lb. to 8,850 lb. per square inch on the core area. The concrete in these columns was the equivalent of a 1: 2: 4 mix. The reported yield point was only slightly

Mr. Wheeler. less than the ultimate strength. Upon the basis of the Joint Committee report the allowable working stress on these columns would not exceed about 1,400 lb. per square inch or 19 per cent of the ultimate strength.

It would seem from the foregoing considerations that the Joint Committee Report is going forward in the art of making better concrete and backward in the working stresses that may be allowed upon it.

To the writer's knowledge there are hundreds of reinforced-concrete buildings in existence in the U. S. A. designed on the basis of the Considere ratio for spirals: 12,000 lb. per square inch in vertical steel and 100 lb. or more on the concrete core area. Some of these buildings have been through severe fires. Others have been grossly overloaded. The quality of the concrete in some is poor and in most of them not equal to the average that should be had for 1: 2: 4 concrete under the committee's specifications. As far as the writer knows there is no indication of distress in any such columns or has there ever been any. Mr. Godfrey referred in the 1921 transactions to the fact that there have been a number of failures of tied columns. Others have noted this fact.

Where then does the Joint Committee find the evidence to justify their column formulas? Why do they consider it necessary to demand a factor of safety of 4 to 5 on spirally hooped and vertically reinforced columns based upon the strength of concrete at the age of 28 days when they know that strength will probably increase by 50 per cent within 4 to 6 months and when they also know that these columns will not receive their full load in less than 3 months after they are poured in the case of any average building in which high column stresses are required? Why are tied columns given such a preference? Why require a factor of safety of 5 or 6 on reinforced concrete which grows stronger with age and a factor of safety of 2 or 3 on steel columns which grow weaker with age.

Referring to the provisions covering design of T-beams. According to the rules laid down, if a building has a girder span of 40 ft. and a clear slab span of 16 ft. and the slab is 6½ in. thick the part of the slab considered as belonging to the T-section of the beam would be 52 in. wide on each side of the beam, but if the span of the beam is reduced to 30 ft. and the beam is 16 in. wide the T could be only 37 in. wide on each side of the beam and if the span reduces to 20 ft. and the beam to 12 in. wide the T reduces to 24 in. wide on each side of the beam or less than half the width for a 40-ft. span. The writer is unable to find any basis in tests or elsewhere for such a rule.

In the matter of the design of flat slabs the writer is certain, based upon his experience and observation as well as upon many records of tests that the committee is far from the truth in the solution of this problem.

In the first place the writer desires to offer certain facts in evidence which he has observed and which others have noticed (see F. R. McMillan in 1921 *Proceedings*, American Concrete Institute). Moderate sags in flat slab floors which gradually manifest themselves even in floors which are not loaded are not evidence of weakness any more than are similar sags

in wood joist construction floors. There is scarcely a wood joist floor to be found that has no sags. The sags in concrete slabs referred to are the result of shrinkage in the concrete and of readjustments in the concrete matrix, resulting from various causes but usually noticeable in concrete which has been mixed and placed wet or overstressed when green by too early removal of forms or settlement of forms. These same phenomena seem to manifest themselves in columns by slight shortening of the columns, as observed by F. R. McMillan and M. B. Lagaard. Mr. Wheeler.

There has been much discussion of flat-slab theory and many tests published. The writer is disappointed that the Joint Committee has been unable to bring about a closer reconciliation between theory and fact than is shown by their report.

Let us take two constructed buildings for example, both of which are ten years old and in neither of which is there any evidence of cracking or any settlements manifest, or weaknesses of any kind. These buildings have been carrying their normal floor loads for some ten years past.

The first building has maximum panels 15 ft. 3½ in. x 8 ft. 0 in. The concrete floor slab is 6½ in. thick. The columns are 16 in. square. The column capitals have a flare of 3 in. and a depth of 4 in. There are no depressions. Part of the building is only two panels wide. There is only one-half as much steel in the slab as is required by the Joint Committee formula. The concrete was of average quality. The building is partitioned for offices above the first floor. The first floor is open so that no support to floors is given by partitions.

This same building designed according to Joint Committee Rules would require a total thickness of slab and drop panel of 11 in. with a drop panel 6 ft. 0 in. x 5 ft. 1 in. x 5 in. and 6-in. slab between drop panels. It would require 100 per cent more steel calculated on the basis of 18,000 lb. per square inch for positive moment than was put into the slab when built.

The other building referred to is two panels wide, one panel being 6 ft. longer than the other. The first floor is open, the upper floors are partitioned. The panels are 22 ft. x 16 ft. The slab is 7½ in. thick. The columns are 16 in. square. The capitals have a flare of 3 in. x 4 in.

This same building designed according to Joint Committee Rules would require a total thickness of 13½ in. for slab and drop panel; drop panels 7 ft. 4 in. x 5 ft. 4 in. x 6 in. thick, with an 8½-in. slab between panels; reinforcing would be about 100 per cent more for positive moment than went into the slab when constructed. The above buildings are two out of many similarly constructed that could be mentioned and that show similar evidence of being designed to meet the requirements.

If the writer correctly understands the formulas of the Joint Committee for four-way slabs without depressions, the compression on the concrete in the last described slab due to negative moment at normal load would equal 1500 lb. per square inch on the outer fiber. The compression due to positive moment in the outer fiber would equal 1190 lb. per square

Mr. Wheeler. inch and the tension in the steel at midspan would equal 39,000 lb. per square inch. The steel used had an ultimate strength of 55,000 lb. to 60,000 lb. per square inch and a yield point around 32,000 lb. per square inch. No compression tests were made on the concrete. The mix was 1:2:4 placed moderately wet. The coarse aggregate was good but the fine aggregate was rather poor so that it is not likely that the concrete would show more than 2,000 lb. per square inch in 28 days. Now if the actual stresses are equal to those determined by Joint Committee formulas there must be some miraculous qualities in a flat slab which not only make it stand up but prevent it from showing any deflections with such stresses existing.

Another building with which the writer is familiar is designed to carry a working live-load of 300 lb. per square foot upon the same basis as the two buildings hereinbefore described, except that the column capitals have a diameter equal to 0.18 of the span, but no drop panels. This building is about 14 years old. It carried for several months of each successive year a working live-load of 425 lb. per square foot over almost the full area of the entire floors of the building. The steel is the same grade as that previously described of low ultimate strength and yield. The concrete is the same mix but the aggregates were better. The floors after all these years of such severe usage hold their shape and are apparently as good as when the building was first constructed. There are a great many buildings and other structures which to the writer's knowledge were similarly designed and have a similar good record of performance. It is certain they would not show this record if the formulas of the committee are correct or approximately correct.

The committee attempts to account for the great strength of flat slabs of some designs as compared with their calculated strength on the assumption that it is the tensile strength of the concrete. The writer has no faith in the truth of this assumption. He believes that the trouble with the Joint Committee's formula is the result of their failure to give the effects of shear and combined twisting and bending proper weight. They have given some credit to these agencies and it is enough for slabs designed on beam-strip theory. The writer sincerely hopes that the committee will make another attempt to solve this problem. Until such time the writer prefers to rely upon the theory of flat slabs as devised by H. T. Eddy and which agrees closely with results actually attained in the field rather than to accept a theory which does not agree with field results and which must be explained by giving credit to the tensile strength of concrete although in Chap. II, Art. 103, Item (e) of the report that tensile strength is assumed to be neglected.

The impression seems to be quite general in the engineering and architectural professions that flat-slab construction is not applicable to light construction buildings. The writer has found the reverse to be true. Properly designed flat slabs are ideal construction for light-load buildings, but unfortunately many of our city building codes are so drawn as to preclude

Mr. Wheeler.

their use. In applying flat-slab construction or any other reinforced-concrete construction or any construction to light-load buildings it is necessary to give consideration to the construction loads which may come upon the floors. Reinforced-concrete slabs which are overstrained during construction or while green can not be relied upon to hold their shape although they may be perfectly satisfactory as to strength. There is a tendency among contractors and workmen to look upon a reinforced-concrete floor much as they do a pavement in the street and to load it accordingly. It is entirely practical to avoid careless overloading of floor slabs during construction, but slabs should be designed for loads which provide for practical construction methods and this cannot be done when such loads as 40 lb. and 50 lb. per square foot are the basis of slab design.

On the other hand it is not necessary to provide for construction loads in the design of columns in the ordinary building. In the writer's experience part of the prejudice against flat-slab construction for light-load buildings and buildings which are to be partitioned can be traced to this damage during construction and it is naturally more manifest in the case of such slabs than other types of construction where slabs are thicker in proportion to span. However, the writer has observed this same difficulty with other forms of construction, such as one-way slabs and joist slabs. It is a matter which can be and should be covered in building codes, and which engineers should provide for in their designs regardless of building code requirements.

With further reference to the use of flat-slab construction in buildings of light construction: The writer has employed it very successfully in a good many bank buildings, office buildings, hotels, apartments, clubs, schools, university buildings, institutions of various kinds, churches, city halls, court houses, state capitol buildings, retail stores and others. In the Middle West the writer has found that for a ten- or twelve-story office building flat-slab construction will save from 15 to 20 c. per square foot of floor area on the first cost of the building when designed according to our methods, and that there is also a substantial saving in the cost of operating and maintaining the building after it is built. For further information on this subject reference is made to page 20 of "Buildings and Building Management," issue of Sep. 4, 1922. Under the proposed Joint Committee specifications it would be impractical to use flat-slab construction for such buildings.

The writer is a firm believer in the principle that successful engineering means securing by engineering designs, maximum economy in construction without sacrificing strength, durability, utility or appearance, and that such engineering is a national asset. On the other hand he believes just as firmly that wasteful engineering designs are not the result of successful engineering and are a national liability. We must get the truth and the whole truth as far as it is humanly possible about the design and construction of reinforced concrete. The writer wishes to again commend the committee and to urge them to further effort.

[*Further pertinent discussion starts on p. 461.—Editor.*]

Reinforced Concrete Building Design and Specifications.

Submitted by Committee E-1.

This committee was requested by the Board of Direction to prepare standard building regulations for the use of reinforced concrete using as a basis, the provision of the Standard Specifications for Concrete and Reinforced Concrete of the Joint Committee Report for 1924.

The committee submits, herewith, a draft of such regulations. This draft was prepared by a sub-committee of five members with the aid of written discussions of the Joint Committee Report from other members of the committee. The draft was thoroughly considered at an all-day and evening session of the full committee on Feb. 23 and a half day's session Feb. 24. During these sessions, there was an attendance of 13 members of the committee.

In the preparation of this draft, the committee found it desirable to omit many items covered by the Joint Committee Report as not being matters proper for consideration in a Building Regulation. It was also necessary to modify the language and greatly condense many other provisions. In still other cases, the committee being unable to agree in all respects with the provisions in the report, made such changes as in its judgment were necessary or desirable. It was also necessary to add certain sections not included in the Joint Committee Report.

The committee submits this report with the request that the members of the Institute and others interested, submit suggestions or criticisms of the proposed regulations. It is the program of the committee to continue the study of this draft and to prepare for submission to the Institute for adoption at its next convention, a Tentative Standard Building Regulation for the use of Reinforced Concrete.

The rules of the Institute require that proposed tentative standards be preprinted and circulated 30 days prior to the convention. This will require that suggestions should be in the hands of the committee chairman not later than Sept. 15, 1925. The membership is urged to send in discussion to aid the committee in the preparation of the tentative standards.

F. R. McMillan, *Chairman.*

PRELIMINARY DRAFT OF PROPOSED STANDARD BUILDING REGULATIONS FOR THE USE OF REINFORCED CONCRETE.

CHAPTER A.

INTRODUCTION.

A-1: *Scope.*—These regulations are drawn to cover the use of reinforced concrete in any structure to be erected under the provisions of the building code of which they form a part. They are intended to supplement the general provisions of the code in order to provide for the proper design and construction of structures of this material. In all matters pertaining to the design and construction where these specific regulations are in conflict with other provisions of the code, these regulations shall govern.

A-2: *Application for Permit.*—Application for permit to erect or alter a reinforced concrete structure shall, in addition to complying with the general requirements of the code, be accompanied by duplicate sets in blueprint form of drawing which shall fully show the essential structural details by plans, elevations, sections and bending diagrams. Plans shall be legibly drawn to an appropriate scale. The principal distances and dimensions shall be accurately shown in figures, and the floor loads and strength of concrete for which the structure is designed shall be clearly indicated. The grade of reinforcing steel to be used, whether plain or deformed bars, and all other information necessary to permit the commissioner of buildings or his authorized representative to determine whether the design conforms with these regulations shall be given.

Each plan and drawing shall bear the seal of the architect or engineer as required by the law in this city or state.

The application for permit shall be made by the owner or his representative and shall be accompanied by a certificate signed and sealed by the architect or engineer responsible for the design stating that the construction shown on the plans and details submitted complies with these regulations.

A-3: *Approved Drawings to be at the Work.*—One set of drawings bearing the approval of the commissioner of buildings shall be returned to the applicant when permit to erect or alter is issued. This shall be kept at the site of the construction work at all times until the acceptance of the structure. The other set approved in like manner shall be kept by the commissioner of buildings for a period of two years after the acceptance of the structure.

A-4: *Changes in Plans.*—If any changes from the approved plans, affecting the strength of the structure or the compliance with any other provision of the code are contemplated at any time, revised blueprints in duplicate showing the revised construction and details, shall be submitted to the commissioner of buildings and his approval secured before such changes are made. The approved revised drawings shall replace those rendered obsolete both at the work and in the office of the commissioner of buildings.

A-5: *Violations of These Regulations.*—Should any portion of the construction violate these regulations or other provisions of the code, the same shall be remedied to comply with the requirements or the construction shall be rated for its safe carrying capacity as computed under these regulations.

A-6: *Acceptance of the Structure.*—Upon the completion of the structure the owner or his representative shall notify the commissioner of buildings who shall make or authorize to be made by his representative, a final inspection and shall issue a certificate of acceptance.

No structure shall be used or occupied until it has been accepted and such certificate issued. The acceptance of the structure in this manner shall not prevent the city from requiring full compliance with these regulations should any violations be subsequently discovered.

A-7: *Posting of Loads.*—The commissioner of buildings shall issue signed placards to be posted on each floor, showing the maximum safe load per square foot which may be placed on the floor. It shall be unlawful to load any such floors or any part thereof to a greater extent than the loads indicate on such placards.

CHAPTER B.

DEFINITIONS.

B-1.—The following definitions give the meaning of certain terms as used in these regulations:

Aggregate.—Inert material which is mixed with portland cement and water to produce concrete; in general, aggregate consists of sand, pebbles, gravel, crushed stone, or similar materials.

Anchorage.—The embedment in concrete of a portion of a reinforcement bar, either straight or with hooks, designed to prevent pulling out or slipping of the bar when subjected to stress. (The anchorage of tension reinforcement in beams includes only the embedded length beyond a point of contra-flexure or of zero moment.)

Column.—An upright compression member the length of which exceeds three times its least lateral dimension.

Column Capital.—An enlargement of the upper end of a reinforced concrete column designed and built to act as a unit with the column and flat slab.

Column Strip.—One of two strips in a flat slab panel each ¼ panel in width, occupying the two quarter panel areas outside of the middle strip. (See *Middle Strip.*)

Composite Column.—A circumferentially reinforced concrete column with a core of structural steel or cast iron which is designed to carry a portion of the load.

Concrete.—A mixture of portland cement, fine aggregate, coarse aggregate and water. (See *Mortar.*)

Consistency.—A general term used to designate the relative plasticity of freshly mixed concrete or mortar.

Dead Load.—The weight of the permanent parts of the structure.

Deformed Bar.—Reinforcement bar with shoulders, lugs, or projections formed integrally with the bar during rolling.

Diagonal Direction.—A direction parallel or approximately parallel to the diagonal of the panel of a flat slab.

Dropped Panel.—The structural portion of a flat slab which is thickened throughout an area surrounding the column capital.

Effective Area of Concrete.—The area of a section of the concrete which lies between the tension reinforcement and the compression surface in a beam or slab.

Effective Area of Reinforcement.—The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between its direction and that for which the effectiveness of the reinforcement is to be determined.

Flat Slab.—A concrete slab having reinforcement bars extending in two or more directions without beams or girders to carry the load to supporting members.

Footing.—A structural unit used to distribute wall or column loads to the foundation materials.

Gravel.—Rounded particles larger than sand resulting from the natural disintegration of rocks. (See *Sand.*)

Laitance.—Extremely fine material of little or no hardness which may collect on the surface of freshly-deposited concrete or mortar, resulting from the use of excess mixing water, and usually recognized by its relatively light color.

Live Load.—Loads and forces other than the dead load.

Middle Strip.—A portion of a flat slab panel one-half panel in width, symmetrical with respect to the panel center line and extending through the panel in the direction in which moments are being considered.

Mortar.—A mixture of portland cement, fine aggregate and water. (See *Concrete.*)

Negative Reinforcement.—Reinforcement so placed as to take tensile stress due to negative bending moment.

Panel Length.—The distance in either rectangular direction between centers of two columns of a panel.

Pedestal.—An upright compression member whose height does not exceed three times its least lateral dimension.

Pedestal Footing.—A column footing projecting less than one-half its depth from the faces of the column on all sides and having a depth not more than three times its least width.

Plain Concrete.—Concrete without metal reinforcement.

Portland Cement.—The product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

Positive Reinforcement.—Reinforcement so placed as to take tensile stress due to positive bending moment.

Principal Design Section.—The vertical sections in a flat slab on which the moments in the rectangular directions are critical. (See Section K-2.)

Ratio of Reinforcement.—The ratio of the effective area of the reinforcement cut by a section of a beam or slab to the effective area of the concrete cut by that section.

Rectangular Direction.—A direction parallel to a side of the panel of a flat slab.

Reinforced Concrete.—Concrete in which metal is embedded in such a manner that the two materials act together in resisting forces.

Sand.—Small grains resulting from the natural disintegration of rocks. (See *Gravel.*)

Screen.—A metal plate with closely spaced circular perforations. (See *Sieve.*)

Sieve.—Woven wire cloth with square openings. (See *Screen.*)

Strut.—A compression member other than a column or pedestal.

Wall Beam.—A reinforced concrete beam which extends from column to column along the outer edge of a wall panel.

CHAPTER C.

Materials and Tests.

C-1: *Tests.*—The tests called for in these regulations when ordered by the commissioner of buildings or his authorized representatives shall be arranged for by the owner or his representative. No responsibility for the expense of these tests shall attach to the Department of Buildings. Such tests shall be made in accordance with the standard method of test covering the particular material under consideration, of the American Society for Testing Materials in effect on the date of the adoption of these regulations.

All such tests shall be made by competent persons approved by the commissioner of buildings and copies of the results shall be kept on file in the office of the commissioner of buildings for a period of two years after the acceptance of the structure.

Tests shall be made on any material entering into concrete or reinforced concrete construction when in the opinion of the commissioner of buildings or his authorized representative, there is any doubt as to its suitability for the purpose.

The commissioner of buildings or his authorized representative shall have the right to require tests of the concrete from time to time to determine whether the materials and methods in use are such as to produce concrete of the necessary quality. Specimens for such tests shall be taken at the place where concrete is being deposited, and shall be taken, cured and tested in accordance with the standards of the American Society for Testing materials in effect on the date of adoption of these regulations.

C-2: *Load Tests.*—The commissioner of buildings or his authorized representative shall have the right to order the test under load of any portion of a completed structure, when the conditions have been such as to leave any doubt as to the adequacy of the structure to serve the purpose for which it was intended. Such tests shall not be required to be made on any concrete construction which is less than 60 days old.

In such tests, the member or portion of the structure under consideration shall be subject to a superimposed load equal to 1½ times the live-load plus ½ of the dead-load. This load shall be left in position for a period of 24 hours before removal. If during the test, or upon removal of the load, the member or portion of structure shows signs of failure, the commissioner of buildings shall have the right to rate the structure, or such portions thereof as are of the same character as the portions tested, for live-load less than that for which it was designed.

The holder of a permit for the construction of any building which, through failure of portions to pass the test, has been rated for a live-load less than that for which it was designed, shall have the right to submit other portions of the building to test and any portions which he can show to be satisfactory for the designed load shall be exempt from the application of the reduced live-load rating. He shall also have the right to retest any portion of the structure, provided sufficient time has elapsed and proper effort been made to correct the defects. If the portions retested prove satisfactory to the commissioner of buildings, the designed live-load rating shall be restored on all portions which have been similarly corrected.

In tests applied to determine the suitability of slab or beam construction, the structure will be considered to have failed to pass the test if within 24 hours after the removal of the load the slabs or beams do not show a recovery of at least 75 per cent of the maximum deflection shown during the 24 hours while under load.

C-3: *Inspection.*—All concrete work shall be inspected by the architect or engineer responsible for its design or by a competent representative responsible to the architect or the engineer. A record shall be kept of such inspection which shall cover the quality and quantity of concrete materials, including water, the mixing and placing of the concrete, and the placing of the reinforcing steel. The inspection record shall also include a complete record of the progress of the work and of the temperatures, when these fall below 40 deg. F., and of the protection given to the concrete while curing. These records shall be available for inspection by the commissioner of buildings at all times during the progress of the work and shall be preserved for two years after the acceptance of the structure.

C-4: *Portland Cement.*—Portland cement shall conform to the Standard Specifications and Tests for Portland Cement (Serial Designation C 9-21) of the American Society for Testing Materials.

C-5: *Concrete Aggregates.*—Concrete aggregates shall consist of natural sands and gravels, crushed rock, air-cooled blast-furnace slag or other

inert materials having clean, uncoated grains of strong and durable minerals and shall meet the approval of the commissioner of buildings. Aggregates containing soft, friable, thin, flaky, elongated or laminated particles totaling more than 3 per cent, or containing shale in excess of 1½ per cent, or silt and crusher dust finer than the No. 100 standard sieve in excess of 2 per cent shall not be used. These percentages shall be based on the weight of the combined aggregate as used in the concrete. When all three groups of these deleterious materials are present in the aggregates, the combined amounts shall not exceed 5 per cent by weight of the combined aggregate. Aggregates shall not contain strong alkali, or organic material which gives a color darker than the standard color when tested in accordance with the standard colorimetric test of the American Society for Testing Materials.

The maximum size of the aggregate shall be not larger than one-fifth of the narrowest dimension between forms of the member for which the concrete is to be used nor larger than three-fourths of the minimum clear spacing between reinforcing bars. By maximum size of aggregate is meant the clear space between the sides of the smallest square opening through which 95 per cent by weight of the material can be passed.

C-6: *Water.*—Water used in mixing concrete shall be clean, and free from injurious amounts of oil, acid, alkali, organic matter or other deleterious substances.

C-7: *Metal Reinforcement.*—Metal reinforcement shall conform to the requirements of the Standard Specifications for Billet Steel, Concrete Reinforcement Bars of Structural or Intermediate Grade (Serial Designation: A15-14) of the American Society for Testing Materials. Hard-grade billet steel meeting the requirements of the above specification (A 15-14) or rail steel meeting the requirements for Rail Steel Concrete Reinforcement Bars (Serial Designation A 16-14) of the American Society for Testing Materials, may be used for bars ¾ in. in size and smaller or for larger sizes where no bending is required. The provision in these specifications for machining deformed bars before testing shall be eliminated.

Metal reinforcement, to receive the rating of "deformed bars" which permits the use of higher bond stresses than for plain bars, shall show a bond strength 25 per cent greater than that shown by plain bars of equivalent cross-sectional area.*

C-8: *Storage of Materials.*—Cement and aggregates shall be stored at the work in a manner to prevent deterioration or the intrusion of foreign matter. Any material which has deteriorated or has been damaged shall be immediately and completely removed from the work.

CHAPTER D.

CONCRETE QUALITY AND PROPORTIONS.

D-1: *Concrete Quality.*—Provisions for the design of structures embodied in these regulations are based on the presumption of concrete of

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* The committee has under consideration a test for bond which it will propose as a basis for judging the bond value of different bars.

certain strength. To produce concrete of the required strength, the proportion of the mixing water to the cement shall be accurately controlled. To obtain the strengths indicated in the following table, the ratio of water to cement shall be in the proportions shown. The strengths indicated represent the minimum ultimate strength in compression which may be expected at 28 days when cured and tested as specified in Section C-1.*

Proportion of Mixing Water to Cement.*

Ultimate Strength Used in Design lb. per sq. in.	Water-Cement Ratio U. S. Gal. of Water Per Sack of Cement.
1500	8¼
2000	7¼
2500	6½
3000	5¾

Water or moisture contained in the aggregates must be included in determining the ratio of water to cement.

All structural drawings and plans submitted for approval shall show the strength of concrete to be used and the water-cement ratio necessary to produce that strength as per this table. Such note indicating the water required shall clearly state that this quantity of water includes that contained in the aggregates.

D-2: *Concrete Proportions and Consistency.*—The proportions of aggregates to cement for concrete of any water-cement ratio shall be such as to produce concrete that will work readily into the corners and angles of the form and around the reinforcement without excessive puddling or spading and without permitting free water to collect on the surface. The combined aggregate shall be of such composition of sizes that when separated by the No. 4 standard sieve, the weight retained on the sieve shall not be less than one-half nor more than two-thirds of the total nor shall the amount of coarse material be such as to produce harshness in placing or honeycombing in the structure. When forms are removed, the faces and corners of the members shall show smooth and sound throughout.

D-3: *Control of Proportions.*—The methods of measuring concrete materials shall be such that the proportion of water to cement can be accurately controlled during the progress of the work and easily checked at any time by the commissioner of buildings or his authorized representative. A tolerance of ¼ gal. of water per sack of cement in any batch of concrete will be allowed provided that the average for any 10 consecutive batches does not show a water content greater than that shown in the table and on plans as specified in Section D-1.

* Attention is called to the fact that these proportions of water to cement are based on a wide range of tests and experience and therefore they may be expected to apply to a majority of cases. However, there may be localities where the available materials are such that different water-cement ratios are required for the strengths indicated. It would be advisable therefore for each municipality to conduct the necessary tests to determine the suitability of these limits. For the purpose of a building code it is recommended that the strengths specified be not over 80 per cent of the average values shown by the tests.

The method of delivering the aggregates to the work and of storing and handling shall be such that the moisture content of the aggregates as they come to the mixer shall not be subject to frequent or unnecessary changes.

CHAPTER E.

MIXING AND PLACING CONCRETE.

E-1: *Mixing.*—The concrete shall be mixed until there is a uniform distribution of the materials and the mass is uniform in color and homogeneous. In machine mixing, only batch mixers shall be used. Each batch shall be mixed at least one minute after all the materials are in the mixer and must be completely discharged before recharging.

E-2: *Cleaning Forms and Equipment.*—Before placing concrete all equipment for mixing and transporting the concrete shall be cleaned, all debris shall be removed from the places to be occupied by the concrete, forms shall be thoroughly wetted (except in freezing weather) or oiled, and clay tile that will be in contact with concrete shall be well drenched (except in freezing weather). Reinforcement shall be thoroughly cleaned and secured in position. Concrete shall not be placed until the forms and reinforcement have been inspected by the architect or engineer responsible for the design.

E-3: *Removal of Water from Excavation.*—Water shall be removed from excavations before concrete is deposited, unless otherwise directed by the commissioner of buildings. Any flow of water into the excavation shall be diverted through proper side drains to a sump, or be removed by other approved methods which will avoid washing the freshly deposited concrete. Water vent pipes and drains shall be filled by grouting or otherwise, after the concrete has thoroughly hardened.

E-4: *Transporting.*—Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable by methods which shall prevent the separation or loss of the ingredients. It shall be deposited as nearly as practicable in its final position to avoid rehandling or flowing. Under no circumstances shall concrete that has partially hardened be deposited in the work.

When concrete is conveyed by chuting, the plant shall be of such size and design as to insure a practically continuous flow in the chute. The slope of the chute shall be such as to allow the concrete to flow without separation of the ingredients. The delivery end of the chute shall be as close as possible to the point of deposit. When the operation is intermittent, the spout shall discharge into a hopper. The chute shall be thoroughly flushed with water before and after each run; the water used for this purpose shall be discharged outside the forms.

E-5: *Placing.*—Concrete shall be thoroughly compacted by puddling with suitable tools. In thin walls or inaccessible portions of the forms where rodding or spading is impracticable, the concrete shall be worked into place by tapping or hammering the forms adjacent to the freshly de-

posited concrete. When necessary, openings shall be provided in the forms to permit the placing of concrete in such a manner as to avoid accumulations of hardened concrete on the forms or reinforcing bars. The concrete shall be thoroughly worked around the reinforcement, around embedded fixtures, and into the corners of the forms.

E-6: *Curing.*—Exposed surfaces of concrete shall be kept moist for a period of at least 7 days after being deposited.

E-7: *Depositing in Cold Weather.*—When depositing concrete at freezing or near freezing temperatures, the concrete shall have a temperature of at least 40 deg. F., but not more than 120 deg. F. The concrete shall be maintained at a temperature of at least 50 deg. F. for not less than 72 hours after placing or until the concrete has thoroughly hardened. When necessary, concrete materials shall be heated before mixing. Dependence shall not be placed on salt or other chemicals for the prevention of freezing.

E-8: *Bonding Fresh and Hardened Concrete.*—Before depositing new concrete on or against concrete which has set, the forms shall be retightened, the surface of the set concrete shall be roughened, cleaned of foreign matter and laitance, and thoroughly wetted but not saturated. To insure excess mortar at the juncture of hardened and newly deposited concrete, the cleaned and wetted surfaces of the hardened concrete, including vertical and inclined surfaces, shall be slushed with a coating of 1: 2 cement mortar against which the new concrete shall be placed before the mortar has attained its initial set.

CHAPTER F.

FORMS AND DETAILS OF CONSTRUCTION.

F-1: *Design of Forms.*—Forms shall conform to the shape, lines and dimensions of the member as called for on the plans.

Forms shall be substantial and sufficiently tight to prevent leakage of mortar; they shall be properly braced or tied together so as to maintain position and shape and insure safety to workmen and passersby. If adequate foundation for shores cannot be secured, trussed supports shall be provided.

Temporary openings shall be provided at the base of column and wall forms, and at other points where necessary, to facilitate cleaning and inspection immediately before depositing concrete.

F-2: *Removal of Forms.*—Forms shall not be disturbed until the concrete has hardened sufficiently to permit their removal with safety. Shoring shall not be removed until the member has acquired sufficient strength to safely support its weight and the load upon it. Members subject to additional loads during construction shall be adequately shored to support both the member and construction loads in such a manner as will protect the member from damage by the loads.

F-3: *Cleaning and Bending Reinforcement.*—Metal reinforcement, before being placed, shall be thoroughly cleaned of loose mill and rust scale

and of other coatings that will destroy or reduce the bond. Reinforcement shall be carefully formed to the dimensions indicated on the plans. Cold bends shall be made around a pin having a diameter of four or more times the least dimension of the bar.

Metal reinforcement shall not be bent or straightened in a manner that will injure the material. Bars with kinks or bends not shown on the plans shall not be used. Heating of reinforcement will be permitted only when approved by the commissioner of buildings.

F-4: *Placing Reinforcement.*—Metal reinforcement shall be accurately placed and secured, and shall be supported by concrete or metal chairs or spacers, or metal hangers. The minimum clear distance between parallel bars shall be 1½ times the diameter for round bars or 1½ times the diagonal for square bars; if the ends of bars are anchored as specified in Sec. J-5, the clear spacing may be made equal to the diameter of round bars or to the diagonal of square bars, but in no case shall the spacing between bars be less than 1 in., nor less than 1 1/3 times the maximum size of the coarse aggregate. Bars at the upper face of any member shall be embedded a clear distance of not less than one diameter.

F-5: *Splices and Offsets in Reinforcement.*—In slabs, beams and girders, splices of reinforcement shall not be made at points of maximum stress without the approval of the commissioner of buildings. Splices, where permitted, shall provide sufficient lap to transfer the stress between bars by bond and shear. In such splices the bars shall be spaced at the minimum distance specified in Sec. F-4.

Splices in columns, piers and struts shall provide sufficient lap to transfer the stress by bond.

Where changes in the cross-section of a compression member occur, the longitudinal bars shall be sloped for the full length of the member or offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion from the axis of the member shall not be more than 1 in 6.

F-6: *Protective Covering of Concrete.*—At the under side of footings metal reinforcement shall have a minimum covering of 3 in. of concrete. At other surfaces of concrete exposed to the ground or weather, metal reinforcement shall be protected by not less than 2 in. of concrete.

In fire-resistive construction, metal reinforcement shall be protected by not less than 1 in. of concrete in slabs and walls, and not less than 1½ in. in beams, girders and columns, provided aggregate showing an expansion not materially greater than that of limestone or trap rock is used; when impracticable to obtain aggregate of this grade, the protective covering shall be ½ in. thicker and shall be reinforced with metal mesh having openings not exceeding 3 in., placed 1 in. from the finished surface.

In structures where the fire hazard is limited, the metal reinforcement shall not be placed nearer the exposed surface than ¾ in. in slabs and walls or 1½ in. in beams, girders and columns.

Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion.

F-7: *Construction Joints.*—Joints not indicated on the plans shall be so made and located as to least impair the strength of the structure. Where a horizontal joint is to be made, any excess water and laitance shall be removed from the surface after concrete is deposited. Before depositing of concrete is resumed the hardened surface shall be treated as specified in Sec. E-8. Column forms shall be filled to a depth of 4 in. with 1 : 2 cement mortar immediately before depositing concrete.

At least two hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs supported thereon. Haunches and column capitals shall be considered as part of and to act continuous with the floor.

Construction joints in floors shall be located near the middle of spans of slabs, beams or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. Provision shall be made for shear by use of inclined reinforcement.

CHAPTER G.

DESIGN—GENERAL CONSIDERATIONS.

G-1: *Assumptions.*—The design of reinforced-concrete members under these specifications shall be based on the following assumptions:

(a) Calculations are made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.

(b) A plane section before bending remains plane after bending, shearing distortions being neglected.

(c) The modulus of elasticity of concrete in compression is constant within the limits of working stresses and the distribution of compressive stress in beams is rectilinear.

(d) The moduli of elasticity of concrete in computations for the position of the neutral axis, for the resisting moment of beams, and for compression of concrete in columns, are as follows:

(1) $\frac{1}{15}$ that of steel, when the compressive strength of the concrete at 28 days exceeds 1,500 and does not exceed 2,200 lb. per sq. in.;

(2) $\frac{1}{12}$ that of steel, when the compressive strength of the concrete at 28 days exceeds 2,200 and does not exceed 2,900 lb. per sq. in.;

(3) $\frac{1}{10}$ that of steel, when the compressive strength of the concrete at 28 days is greater than 2,900 lb. per sq. in.

(e) In calculating the moment of resistance of reinforced-concrete beams and slabs the tensile resistance of the concrete is neglected.

(f) The bond between the concrete and the metal reinforcement remains unbroken throughout the range of working stresses. Under compression the two materials are therefore stressed in proportion of their moduli of elasticity.

(g) Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected, except in the design of reinforced-concrete columns, where the effect of shrinkage stresses has been given consideration in the design formulas and allowable working stresses.

G-2: *Notation.*—The symbols and notation used in these regulations are defined as follows:

a = width of face of column or pedestal;

α = angle between inclined web bars and axis of beam;

A = total net area of column, footing, or pedestal, exclusive of fireproofing;

A' = loaded area of pedestal, pier or footing;

A_c = $A\ (1 - p)$ = net area of concrete core of column (core area minus reinforcement);

A'_c = net area of concrete in columns with lateral ties (total column area minus area of reinforcement);

A_s = effective cross-sectional area of metal reinforcement in tension in beams or compression in columns; and the effective cross-sectional area of metal reinforcement which crosses any of the principal design sections of a flat slab and which meets the requirements of Sec. K-12;

A_v = total area of web reinforcement in tension within a distance of s (s_1, s_2, s_3, etc.) or the total area of all bars bent up in any one plane;

b = width of rectangular beam or width of flange of T-beam;

b_1 = dimension of the dropped panel of a flat slab in the direction parallel to l_1;

c = base diameter of the largest right circular cone which lies entirely within the column (including the capital) whose vertex angle is 90 deg. and whose base is 1½ in. below the bottom of the slab or the bottom of the dropped panel;

c = projection of footing from face of column;

d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement;

E_c = modulus of elasticity of concrete in compression;

E_s = modulus of elasticity of steel in tension = 30,000,000 lb. per sq. in.;

f_c = compressive unit stress in extreme fiber of concrete;

f'_c = ultimate compressive strength of concrete at age of 28 days, based on tests of 6 by 12-in. or 8 by 16-in. cylinders made and tested in accordance with the Standard Methods of Making and Storing Specimens of Concrete in the Field and the Tentative Methods of Making Compression Tests of Concrete, of the A. S. T. M.

f_r = compressive unit stress in metal core;

f_s = tensile unit stress in longitudinal reinforcement;

f_v = tensile unit stress in web reinforcement;

F = total tension in a bar;

F' = total tensile stress in a bar developed in the length "y." See Sec. J-3.

h = unsupported length of column;

I = moment of inertia of a section about the neutral axis for bending;

l = span length of beam or slab (generally distance from center to center of supports; for special cases, see Sec. H-3 and K-6);

l = span length of flat slab, center to center of columns, in the rectangular direction in which moments are considered (see Sec. K-3);

l_1 = span length of flat slab, center to center of columns, perpendicular to the rectangular direction in which moments are considered;

M = bending moment or moment of resistance in general;

M_o = sum of positive and negative bending moments in either rectangular direction, at the principal design sections of a panel of a flat slab;

n = E_s/E_c = ratio of modulus of elasticity of steel to that of concrete;

Σo = sum of perimeters of bars in one set;

p = ratio of effective area of tension reinforcement to effective area of concrete in beams = A_s/bd; and the ratio of effective area of longitudinal reinforcement to the area of the concrete core in columns;

P = total safe axial load on column whose h/R is less than 40;

P' = total safe axial load on long column;

r_a = permissible working stress in concrete over the loaded area of a pedestal, pier or footing;

R = ratio of positive or negative moment in two column strips or one middle strip of a flat slab, to M_o;

R = least radius of gyration of a section;

s = spacing of web members, measured at the mid-depth of the beam and in the direction of the longitudinal axis of the beam;

t = thickness of flange of T-beam;

t_1 = thickness of flat slab without dropped panels or thickness of a dropped panel;

t_2 = thickness of flat slab with dropped panels at points away from the dropped panel;

u = bond stress per unit of area of surface of bar;

v = shearing unit stress;

V = total shear;

w = uniformly distributed load per unit of length of beam or slab;

w = upward reaction per unit of area of base of footing;

w' = uniformly distributed dead- and live-load per unit of area of a floor or roof;

W = total dead- and live-load uniformly distributed over a single panel area;

x = length of bar added for anchorage, including the hook, if any;

y = distance from the point at which the tension is computed to the point of beginning of anchorage.

G-3: *Unit Stresses in Terms of Ultimate Strength of Concrete.*—As specified in Sec. D-1, the structural drawings and plans shall show the ultimate strength of concrete for which the several portions of the structures were designed. The working stresses to be used in the computations shall be based on the ultimate strength indicated on the drawings and in the ratios shown in the succeeding sections of these regulations. The ultimate strength is designated as f'_c and refers to the ultimate strength at 28 days, based on 6 by 12-in. or 8 by 16-in. cylinders made, cured and tested in accordance with the Standard Methods of Making and Storing Specimens of Concrete in the Field and Tentative Methods of Making Compression Tests, of the American Society for Testing Materials in effect at the time of adoption of these regulations. These working stresses in the concrete are here summarized together with the permissible working stresses in the reinforcement.

SUMMARY OF WORKING STRESSES.

Direct Stresses in Concrete.

In columns; varies, see Sections L-3 and L-4.
In long columns; see Section L-8.
In piers and pedestals .... $0.25f'_c$

Flexural Stresses in Concrete.

Extreme fibre stress in compression in flexure .... $0.40f'_c$
Extreme fibre stress in compression in flexure adjacent to supports of continuous beams .... $0.45f'_c$

Shearing Stresses in Concrete.

Beams without special anchorage of longitudinal reinforcement:
- With no web reinforcement .... $0.02f'_c$
- With stirrups or bent-up bars or combinations of the two .... $0.02f'_c$ to $0.06f'_c$

Beams with special anchorage of longitudinal reinforcement:
- With no web reinforcement .... $0.03f'_c$
- With stirrups or bent-up bars or combinations of the two .... $0.03f'_c$ to $0.12f'_c$

In flat slabs:
- At distance d from edge of column cap or dropped panel, see Sec. I-6 .... $0.025f'_c$ to $0.03f'_c$

In footings:

Where longitudinal bars have no special anchorage.. $0.02f'_c$
Where longitudinal bars have special anchorage..... $0.03f'_c$

Bond Between Concrete and Reinforcement.

In beams and slabs, plain bars	$0.04f'_c$
In beams and slabs, deformed bars	$0.05f'_c$
In footings, plain bars, one way	$0.04f'_c$
In footings, plain bars, two ways	$0.03f'_c$
In footings, deformed bars, one way	$0.05f'_c$
In footings, deformed bars, two ways	$0.0375f'_c$

Stresses in Reinforcement.

Tension

(a) Billet-steel bars:	
(1) Structural steel grade	16,000 lb. per sq. in.
(2) Intermediate grade	18,000 " " " "
(3) Hard grade (where permitted)	18,000 " " " "
(b) Rail-steel bars (where permitted)	18,000 " " " "
(c) Structural steel shapes	16,000 " " " "
(d) Other steel reinforcement, 45 per cent of the yield point stress but not to exceed	18,000 " " " "

Compression

(a) Bars	nf_c
(b) Structural steel core of composite column	16,000 lb. per sq. in.
reduced for slenderness ratio	(See Section L-6)
(c) Structural steel column	16,000 lb. per sq. in.
reduced for slenderness ratio	(See Section L-7)
Composite cast-iron column	10,000 lb. per sq. in.
reduced for slenderness ratio	(See Section L-6)

CHAPTER H.

FLEXURAL COMPUTATIONS AND MOMENT COEFFICIENTS.

H-1: *Formulas for Flexure.*—Computations of flexural resistance of reinforced-concrete beams and slabs shall be based on the assumptions of Sec. G-1. The customary formulas or their equivalent shall be used.

H-2: *Working Stresses in Flexure.*—See Sec. G-3.

H-3: *Span Length.*—The span length of freely supported beams and slabs shall be the clear span plus the depth of beam or slab but shall not exceed the distance between centers of the supports.

The span length for continuous or restrained beams built to act integrally with supports shall be the clear distance between faces of supports.

For continuous or restrained beams having brackets built to act integrally with both beam and support and of a width not less than the width of the beam and making an angle of 45 deg. or more with the horizontal the span shall be measured from the section where the combined

depth of the beam and bracket is at least one-third more than the depth of the beam. No portion of such a bracket shall be considered as adding to the effective depth of the beam.

Maximum negative moments are to be considered as existing at the ends of the span, as defined above.

H-4: *Unsupported Length of Beams.*—The distance between lateral supports of the compression area of a beam shall not exceed 24 times the least width of compression flange.

H-5: *Requirements for T-Beams.*—In T-beam construction the slab shall be built integral with the beam. The effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth of the span length of the beam, and its overhanging width on either side of the web shall not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

For beams having a flange on one side only, the effective flange width to be used in design shall not exceed one-tenth of the span length of the beam, and its overhanging width from the face of the web shall not exceed six times the thickness of the slab, nor one-half the clear distance to the next beam.

Where the principal slab reinforcement is parallel to the beam, transverse reinforcement, not less in amount than 0.3 per cent of the sectional area of the slab, shall be provided in the top of the slab and shall extend across the beam and into the slab not less than one-fourth of the clear span of the slab measured parallel to the beam. The spacing of the bars shall not exceed 18 in.

Provision shall be made for the compressive stress at the support in continuous T-beam construction.

The overhanging portion of the flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

Isolated beams in which the T-form is used only for purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web and a total flange width not more than four times the web thickness.

H-6: *Moment Coefficients for the Usual Conditions.*—(A) **Freely supported or slightly restrained continuous beams or slabs of equal span; uniform load.** Beams and slabs of equal spans freely supported or built to act integrally with beams, girders or other slightly restraining support, or beams and slabs built into brick or masonry walls in a manner which develops only partial end restraint, and carrying uniformly distributed loads shall be designed for the following moments at critical sections:

(*a*) Beams and slabs of one span,

Maximum positive moment near center,

$$M = \frac{wl^2}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

(*b*) Beams and slabs continuous for two spans only,

(1) Maximum positive moment near center,

$$M = \frac{wl^2}{10} \qquad \text{(2)}$$

(2) Negative moment over interior support,

$$M = \frac{wl^2}{8} \qquad \text{(3)}$$

(*c*) Beams and slabs continuous for more than two spans,

(1) Maximum positive moment near center and negative moment at support of interior spans,

$$M = \frac{wl^2}{12} \qquad \text{(4)}$$

(2) Maximum positive moment near centers of end spans and negative moment at first interior support,

$$M = \frac{wl^2}{10} \qquad \text{(5)}$$

(*d*) Negative moment at end supports for cases (*a*), (*b*), (*c*) of this section,

$$M = \text{not less than } \frac{wl^2}{16} \qquad \text{(6)}$$

(B) Fully restrained continuous beams or slabs of equal span: uniform load.

Beams and slabs of equal span built to act integrally with columns, walls, or other restraining supports and assumed to carry uniformly distributed loads, shall (except as provided in (A)) be designed for the following moments at critical sections:

(*a*) Interior spans,

(1) Negative moment at interior supports except the first,

$$M = \frac{wl^2}{12} \qquad \text{(7)}$$

(2) Maximum positive moment near centers of interior spans,

$$M = \frac{wl^2}{16} \qquad \text{(8)}$$

(*b*) End spans of continuous beams and slabs and beams and slabs of which I/l is less than twice the sum of the values of I/h for the exterior columns above and below which are built into the beams:

(1) Maximum positive moment near center of span and negative moment at first interior supports,

$$M = \frac{wl^2}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

(2) Negative moment at exterior supports,

$$M = \frac{wl^2}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

(c) End spans of continuous beams, and beams of one span, in which I/l is equal to or greater than twice the sum of the values of I/h for the exterior column above and below which are built into the beams:

(1) Maximum positive moment near center of span and negative moment at first interior support,

$$M = \frac{wl^2}{10} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

(2) Negative moment at exterior support,

$$M = \frac{wl^2}{16} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

In (B) and (C) "I" represents the moment of inertia which, for those calculations, shall be computed on the assumption that the member is homogeneous, neglecting the reinforcement but including that portion of the concrete section outside of the reinforcement which is ordinarily considered as fireproofing. l and h are the span length and column height respectively as defined in Section H-3 and L-2.

(C) Continuous beams or slabs of unequal span or with non-uniform loads.

Continuous beams with unequal spans, or with other than uniformly distributed loading, whether freely-supported or restrained, shall be designed for the actual moments under the conditions of loading and restraint.

Provision shall be made where necessary for negative moment near the center of short spans which are adjacent to long spans, and for the negative moment at the end supports, if restrained.

CHAPTER I.

SHEAR AND DIAGONAL TENSION.

I-1: *Shearing Unit Stress.*—Shearing stresses themselves are seldom important in the design of beams or slabs. However, since they are a convenient measure of diagonal tension which must be provided for, the provisions of this chapter are expressed in terms of shear. The shearing unit stress, v, in reinforced-concrete beams shall be computed by Formula 14:

$$v = \frac{8\,V}{7\,bd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

The shearing unit stress shall be computed on the minimum width of rectangular beams and on the minimum thickness of the web in beams of I- or T-section.

In tile and joist construction, the shearing unit stress shall be computed on a width equal to the thickness of the concrete web plus the thickness of the vertical webs of the concrete or clay tile in contact with the joist.

I-2: *Types of Web Reinforcement.*—Web reinforcement may consist of:

(a) Vertical stirrups or web reinforcing bars;

(b) Inclined stirrups or web reinforcing bars forming an angle of 30 deg. or more with the axis of the beam.

(c) Longitudinal bars bent up at an angle of 15 deg. or more with the axis of the beam.

Stirrups or bent up bars to be considered effective as web reinforcement shall be anchored at both ends, according to the provisions of Sec. J-5.

I-3: *Spacing of Stirrups or Bent-Up Bars.*—Where the shearing stress is not greater than $0.06f'_c$ the distance s measured in the direction of the axis of the beam between two successive stirrups or bent-up bars, shall not exceed the value given by Formula 15,

$$s = \frac{45\,d}{a} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

where a is the angle in degrees between the inclined web bars and the axis of the beam.

Bent-up bars and stirrups shall be considered effective in reinforcing the web only within the area between two vertical planes distant $s/2$ in either direction from the point where the bent-up bar crosses the mid-depth of the beam.

Where the shearing stress is greater than $0.06f'_c$ the distance s shall not be greater than two-thirds of the values given by Formula 15.

I-4: *Maximum Shearing Unit Stress in Beams.*—In beams in which the longitudinal reinforcement is without special anchorage, the shearing unit stress computed by Formula 14 shall not exceed the value given by Formula 16 and in no case shall it exceed $0.06f'_c$.

$$v = 0.02f'_c + \frac{f_v A_v}{bs}\left(\sin a + \cos a\right) \ldots\ldots\ldots\ldots (16)$$

In beams in which the longitudinal reinforcement is anchored as specified in Sec. J-5, the shearing unit stress shall not exceed the value given by Formula 16 when $0.03f'_c$ is substituted for $0.02f'_c$, and in no case shall it exceed $0.12f'_c$.

Where the entire web reinforcement consists of longitudinal bars bent up in a single plane the allowance for the quantity $\frac{f_v A_v}{bs}(\sin a + \cos a)$

in Formula 16 shall not exceed 75. Such bent-up reinforcement shall cross the mid-depth of the beam at a distance not greater than $s/2$ from the face of the support and shall not be considered effective in resisting diagonal tension over a distance greater than s from the support, where s has the value by Formula 15 of Sec. I-3.

I-5: *Combined Web Reinforcement.*—Where two or more types of web reinforcement are used in conjunction, the total shearing resistance of the beam shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete (the term $0.02f'_c$ or $0.03f'_c$ in Formula 16) shall be included only once. In no case shall the maximum shearing stress be greater than the limiting values in Sec. I-4.

I-6: *Shearing Stress in Flat Slabs.*—In flat slabs, the shearing unit stress computed by Formula 14 (in which d shall be taken as $t_1 - 1\frac{1}{2}$) on a vertical section which lies at a distance $t_1 - 1\frac{1}{2}$ from the edge of the column capital and parallel with it shall not exceed $0.02f'_c$ multiplied by the following factor: 1 plus the ratio which the cross-sectional area of the negative reinforcement in the width of strip directly above the column capital bears to the cross-sectional area of the negative reinforcement in the full width of two column strips. At least 25 per cent of the total cross-sectional area of the negative reinforcement in two column strips must be within the width of strip directly above the column capital.

In no case shall the unit shearing stress exceed $0.03f'_c$.

The shearing unit stress computed by Formula 14 (in which d shall be taken as $t_2 - 1\frac{1}{2}$) on a vertical section which lies at a distance of $t_2 - 1\frac{1}{2}$ from the edge of the dropped panel and parallel with it shall not exceed $0.03f'_c$. At least 50 per cent of the cross-sectional area of the negative reinforcement in two column strips must be within the width of strip directly above the dropped panel.

I-7: *Shear and Diagonal Tension in Footings.*—The shearing stress shall be taken as not less than that computed by Formula 14. The stress on the critical section shall not exceed $0.02f'_c$ for footings with straight bars, nor $0.03f'_c$ for footings in which the bars are anchored at both ends by adequate hooks or otherwise as specified in Sec. J-5.

The critical section for diagonal tension in footings on soil shall be computed on a vertical section through the perimeter of the lower base of a frustum of a cone or pyramid which has a base angle of 45 deg., and which has for its top the base of the column or pedestal and for its lower base the plane at the centroid of longitudinal reinforcement.

The critical section for diagonal tension in footings on piles shall be computed on a vertical section at the inner edge of the first row of piles entirely outside a section midway between the face of the column or pedestal and the section defined for soil footings, but in no case outside of that section. For piles not arranged in rows, the critical section shall be taken midway between the face of the column and the section defined for soil footings.

CHAPTER J

BOND AND ANCHORAGE.

J-1: *Computation of Bond Stress in Beams.*—Where bar reinforcement is used to resist tensile stresses developed by beam action, the bond stress shall be taken as not less than that computed by Formula 17.

$$u = \frac{8\,V}{7\,\Sigma od} \qquad \text{. . . . . (17)}$$

For continuous or restrained members, the critical section for bond for the positive reinforcement shall be assumed to be at the point of inflection, that for the negative reinforcement shall be assumed to be at the face of the support, and at the point of inflection. For simple beams or at the outer ends of freely supported end spans of continuous beams, the critical section for bond shall be assumed to be at the face of the support.

Bent-up longitudinal bars which, at the critical section, are within a distance $\frac{d}{3}$ from horizontal reinforcement under consideration may be included with the straight bars in computing $\sum o$.

In footings only the bars specified in Sec. M-4 as effective in resisting bending moment shall be considered as resisting bond stresses. Special investigation shall be made of bond stresses in footings with stepped or sloping upper surface, as maximum bond stresses may occur at the vertical plane of the steps or near the edges of the footing.

J-2: *Permissible Bond Stress: Ordinary Anchorage.*—In beams where the ordinary anchorage described in Sec. J-4 is provided, the bond stress computed by Formula 17 at any section shall not exceed the following values:

For plain bars . . . . $u = 0.04f'_c$
For deformed bars meeting the requirements of
Section C-7 . . . . $u = 0.05f'_c$

The permissible bond stress for footings and similar members in which reinforcement is placed in more than one direction shall not exceed 75 per cent of the above values. Small percentages of reinforcement added for temperature or shrinkage stresses shall not be interpreted as requiring this reduction.

J-3: *Permissible Bond Stress: Special Anchorage.*—In members in bending, bond stresses (computed by Formula 17) exceeding those specified in Sec. J-2, but in no case more than 2½ times the latter, may be used, provided that sufficient additional length of bar is added beyond the theoretical point of zero moment (end of span or point of inflection) to provide for the development of the excess in bond stress over that specified in Sec. J-2. The length x to be added for this purpose may be expressed algebraically by Formula 18.

$$xu \sum o = F - F' \qquad \text{. . . . . (18)}$$

where, x = the length of bar added for anchorage, including the hook, if any,
u = permissible bond stress specified in Sec. J-2,
Σo = the perimeter of the bar or bars under consideration,
F = total tension in the bar or bars under consideration,
F' = the total tension in the bar which would be developed in the length y by the computed bond stresses, except that no values greater than those specified in Sec. J-2 be used in the computation, F' = bond stress times oy.
y = the distance from the point at which the tension is computed to the point of beginning of anchorage.

The point of beginning of anchorage shall be taken at the edge of support for freely supported beams, and at the point of inflection (for the loading condition under consideration) for fixed or continuous beams; anchorage of negative reinforcing to be toward the center of the beams from this point.

The length of bar added for anchorage may be either straight or bent. The radius of bend shall not be less than four bar diameters.

J-4: *Ordinary Anchorage Requirements.*—In continuous, restrained or cantilever beams, the length of anchorage x of the tensile negative reinforcement beyond the face of the support shall provide for the full maximum tension by Formula 18 in which u equals the value given in Sec. J-2, and, for this case, since $y = o$, $F' = o$. Such anchorage shall provide a length of bar not less than the depth of the beam. In the case of end supports which have a width less than three-fourths of the depth of the beam, the bars shall be bent down toward the support a distance not less than the effective depth of the beam. The portion of the bar so bent down shall be as near to the end of the beam as protective covering permits. In continuous or restrained beams, negative reinforcement shall be carried to or beyond the point of inflection. Not less than one-fourth of the area of the positive reinforcement shall extend into the support to provide an embedment of ten or more bar diameters.

In simple beams or at the outer ends of freely supported end spans of continuous beams at least one-fourth of the area of the tensile reinforcement shall extend along the tension side of the beam and beyond the face of the support to provide an embedment of ten or more bar diameters.

J-5: *Special Anchorage Requirements.*—Where increased shearing stresses are used as provided in Secs. I-3 and I-6 or increased bond stresses as provided in Sec. J-3, special anchorage of all reinforcement in addition to that required in Sec. J-4 shall be provided as follows:

(a) In continuous and restrained beams, anchorage beyond points of inflection of at least one-third the area of the negative reinforcement and beyond the face of the support of at least one-third the area of the positive reinforcement, shall be provided to develop one-third of the maximum working stress in tension. The anchorage length x shall be computed by

Formula 18 ($y = o$, therefore $F' = 0$) with bond stresses not greater than those specified in Sec. J-2.

(b) At the edges of footings, anchorage for all the bars for one-third the maximum working stress in tension shall be provided within a region where the tension in the concrete, computed as an unreinforced beam, does not exceed 40 lb. per sq. in. In any case the reinforcement bars shall extend to within 4 in. of the edge of the footing but not closer than 3 in. as specified in Sec. F-6.

(c) In simple beams or at the outer ends of freely supported end spans of continuous beams, at least one-half of the tensile reinforcement shall extend along the tension side of the beam to provide an anchorage beyond the face of the support for one-third of the maximum working stress in tension.

J-6: *Anchorage of Web Reinforcement.*—Web bars shall be anchored at both ends by:

(a) providing continuity with the longitudinal reinforcement; or
(b) bending around the longitudinal bar; or
(c) a semi-circular hook which has a radius not less than four times the diameter of the web bar.

Stirrup anchorage shall be so provided in the compression and tension regions of a beam as to permit the development of safe working tensile stress in the stirrup at a point $0.3d$ from either face. (Generally a properly-anchored stirrup whose diameter does not exceed one-fiftieth of the depth of the beam will meet these requirements.)

The end anchorage of a web member not in bearing on the longitudinal reinforcement shall be such as to engage an amount of concrete sufficient to prevent the bar from pulling out. In all cases the stirrups shall be carried as close to the upper and lower surfaces as fireproofing requirements permit.

CHAPTER K

FLAT SLABS.

(Two-Way and Four-Way Systems with Square or Rectangular Panels.)

K-1: *Limitations.*—The term flat slabs as used in these regulations, refers to concrete slabs, having reinforcement bars extending in two or four directions, without beams or girders to carry the load to supporting members. The moment coefficients, moment distribution, and slab thicknesses specified herein are for slabs which have three or more rows of panels in each direction, and in which the panels are approximately uniform in size. Slabs with paneled ceiling or with depressed paneling in the floor shall be considered as coming under the requirements herein given, provided the depth of the thicker portions of the slab does not exceed 1.5 times the depth of the remainder of the slab.

These regulations shall not apply to flat slabs in which the ratio of length to width of panel exceeds 1.4.

K-2: *Panel Strips and Principal Design Section.*—For convenience of reference, a flat slab panel shall be considered as consisting of strips as follows:

A *middle strip* one-half panel in width, symmetrical with respect to the panel center line and extending through the panel in the direction in which moments are being considered;

Two *column strips*, each one-quarter panel in width, occupying the two quarter panel areas outside of the middle strip.

When considering moments in the direction of the width of the panel, the panel is similarly divided by strips, the widths of which are respectively one-half and one-quarter of the length of the panel.

In the succeeding paragraphs, the provisions for limiting moments, etc., are related to certain critical sections. These sections are referred to as the principal design sections and are located as follows:

Sections for Negative Moment. These shall be taken along the edges of the panel, that is, along the lines joining the column centers. For the column strips, the section shall follow the center line between columns to the edge of the column capital (i. e., to a point $c/2$ from the column center) and then around the circumference of the column capital for a one-quarter circumference.

Sections for Positive Moment. These shall be taken on the center-line of the panel, crossing the strips for which moments are being considered.

K-3: *Moments in Interior Panels.*—In flat slabs in which the ratio of reinforcement (p) for negative moment in the column strip is not greater than 0.01, the numerical sum of the positive and negative moments in the direction of either side of a rectangular panel shall be not less than that given by Formula 19.

$$M_o = 0.09\ Wl\left(1 - \frac{2c}{3l}\right)^2 \qquad (19)$$

where M_o = sum of positive and negative bending moments, at the principal design sections, in the direction in which the length is given by l. This moment is in foot-pounds where the other items are in the units indicated below;

c = base diameter in feet of the largest right circular cone which lies entirely within the column (including the capital) the vertex angle of which is 90 deg. and the base of which is 1½ in. below the bottom of the slab or the bottom of the dropped panel;

l = span length in feet of the flat slab panel, center to center of columns in the direction in which moments are considered. (When considering moments in any direction, the width of column and middle strips must be related to the length of span at right angles to that in which moments are being considered.)

W = total dead- and live-load in pounds uniformly distributed over a single panel area.

K-4: *Moments in Principal Design Sections.*—The moments in the principal design sections shall be those given in the accompanying table of moments, except as follows:

(a) The sum of the maximum negative moments in the two column strips may be greater or less than the values given in table of moments by not more than 0.03 M_o.

(b) The maximum negative moment and the maximum positive moments in the middle strip and the sum of the maximum positive moments in the two-column strips may be greater or less than the values given in table of moments by not more than 0.01 M_o.

MOMENTS TO BE USED IN DESIGN OF FLAT SLABS.
For Interior Panels Fully Continuous.

Strip	Flat Slabs without Dropped Panels Negative	Flat Slabs without Dropped Panels Positive	Flat Slabs with Dropped Panels Negative	Flat Slabs with Dropped Panels Positive
Slabs with 2-way Reinforcement.				
Column strip	0.23 M_o	0.11 M_o	0.25 M_o	0.10 M_o
2-Column strips	0.46 M_o	0.22 M_o	0.50 M_o	0.20 M_o
Middle strip	0.16 M_o	0.16 M_o	0.15 M_o	0.15 M_o
Slabs with 4-way Reinforcement.				
Column strip	0.25 M_o	0.10 M_o	0.27 M_o	0.095 M_o
2-Column strips	0.50 M_o	0.20 M_o	0.54 M_o	0.190 M_o
Middle strip	0.10 M_o	0.20 M_o	0.08 M_o	0.190 M_o

K-5: *Lateral Dimensions of Dropped Panels.*—The dropped panel shall have a length or diameter in each direction parallel to a side of the panel of not less than one-third the panel length in that direction.

K-6: *Thickness of Slabs and Dropped Panels.*—The total thickness of the slab through the dropped panel, t_1, in inches, or of the slab if a dropped panel is not used, shall be not less than the value given by Formula 20.

$$t_1 = 0.038\left(1 - 1.44\frac{c}{l}\right) l \sqrt{Rw'\frac{l_1}{b_1}} + 1\tfrac{1}{2} \qquad (20)$$

where R = ratio of negative moment in the two-column strips parallel to the length to the total moment M_o;

w' = uniformly distributed dead- and live-load per sq ft.;

l_1 = width in feet of the panel at right angles to the direction of the length l;

b_1 = dimension in feet of the dropped panel in the direction parallel to l_1, except that in a slab without dropped panel b_1 shall be taken as 0.5 l_1.

For slabs with dropped panels the total thickness in inches at points beyond the dropped panel shall be not less than

$$t_2 = 0.02\, l \sqrt{w'} + 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

The dropped panel shall have a thickness, t_1, not greater than 1.5 t_2.

In determining minimum thickness by Formulas 20 and 21, the value of l shall be the panel length center to center of the columns, on the long side of panel, and the value of l_1 shall be the panel width, center to center ot the columns.

The slab thickness t_1 or t_2 shall in no case be less than $l/32$ for floor slabs, and not less than $l/40$ for roof slabs.

K-7: *Wall and Other Irregular Panels.*—In wall panels and other panels in which the slab is not continuous with an adjacent panel, the maximum negative moment at the edge of the panel opposite to the discontinuous edge and the maximum positive moment at the center of this panel shall be increased as follows:

(a) In the column strip perpendicular to the wall or discontinuous edge, 15 per cent greater than that given in the table of moments for interior panels;

(b) Middle strip perpendicular to wall or discontinuous edge, 30 per cent greater than that given in the table of moments for interior panels.

In these strips the bars used for positive moments perpendicular to the discontinuous edge shall extend to the edge of the panel at which the slab is discontinuous.

At the wall or discontinuous edge the negative moment in the column strip shall be taken as not less than 90 per cent and in the middle strip not less than 65 per cent of the corresponding moments for a normal interior panel as given in the table of Sec. K-4.

K-8: *Panels with Marginal Beams.*—In panels having a marginal beam on one edge or on each of two adjacent edges, the beam shall be designed to carry at least the load superimposed directly upon it, exclusive of the panel load. A marginal beam which has a depth greater than the thickness of the dropped panel into which it frames, shall be designed to carry, in addition to the load superimposed directly upon it, a uniformly distributed load equal to at least one-fourth of the total live- and dead-load for which the adjacent panel or panels are designed. Slabs supported by marginal beams on opposite edges shall be designed as freely supported slabs for the entire load.

Column strips adjacent to and parallel with marginal beams having a depth less than the thickness of the dropped panel, shall be designed to resist the moment specified for a column strip in the table of moments. Column strips adjacent to and parallel with marginal beams having a depth greater than the thickness of the dropped panel, shall be designed

to resist a moment at least one-half as great as that specified for a column strip in the table of moments.

In wall columns where brackets are used in place of capitals, the value of c in the direction in which the bracket extends, shall be taken as twice the distance from the center of the column to a point $1\frac{1}{2}$ in. back from the edge of the bracket and averaged with the value of c for an interior column capital in the computation for moment in Formula 19. The value of c for column strips parallel and adjacent to marginal beams shall be taken as equal to the width of the wall column if no bracket is used in this direction.

K-9: *Flat Slabs on Bearing Walls.*—Where there is a beam or a bearing wall at the center line of columns in the interior portion of a continuous flat slab, the negative moment at the beam or wall line in the middle strip perpendicular to the beam or wall shall be taken as 30 per cent greater than the negative moment specified in the table of moments (Sec. K-4) for a middle strip. The column strip adjacent to and lying on either side of the beam or wall shall be designed to resist moments at least one-half of those specified in the table of moments (Sec. K-4) for a column strip.

K-10: *Point of Inflection.*—In the middle strip the point of inflection for the slabs without dropped panels shall be assumed at a line $0.30\,l$ distant from the center of the span and for slabs with dropped panels $0.25\,l$ distant from the center of the span.

In the column strip the point of inflection for slabs without dropped panels shall be at a line $0.30\,(l - c)$ distant from the center of the panel and $0.25\,(l - c)$ for slabs with dropped panels.

K-11: *Effective Reinforcement.*—The reinforcement which crosses any section and which fulfills the requirements given in Sec. K-12 may be considered as effective in resisting the moment at the section. The sectional area of a bar multiplied by the cosine of the angle between the direction of the axis of the bar and any other direction may be considered effective as reinforcement in that direction.

K-12: *Arrangement of Reinforcement.*—Provision shall be made for securing the reinforcement in place so as to resist properly not only the critical moments, but also the moments at intermediate sections. Provision shall also be made for possible shifting of the point of inflection by carrying all bars in rectangular or diagonal directions, to points at least 20 diameters beyond the point of inflection each side of a section of critical moment, either positive or negative. Lapped splices shall not be permitted at or near regions of maximum stress except as described in Sec. F-5. At least four-tenths of all bars in each direction shall be of such length and shall be so placed as to provide reinforcement at two sections of critical negative moment and at the intermediate section of critical positive moment. Not less than one-third of the bars used for positive reinforcement in the column strip shall extend into the dropped panel at least 20 diameters of the bar, or in case no dropped panel is used, shall extend to within

one-eighth of the span length from the center line of the column or the support.

K-13: *Special Panel Arrangement.*—For structures having a width of less than three (3) rows of panels, or in which irregular or special panels are used, an analysis shall be made of the moments developed in both slabs and columns. When so required, computations shall be submitted to the commissioner of buildings for approval.

CHAPTER L.

REINFORCED CONCRETE COLUMNS.

L-1: *Limiting Dimensions.*—Unless designed as long columns under the provisions of Sec. L-8, reinforced concrete columns shall not be longer than 40 times the least radius of gyration ($40R$). Principal columns in buildings shall have a minimum diameter or thickness of 12 in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of 6 in.

L-2: *Unsupported Length and Radius of Gyration of Columns.*—The unsupported length of reinforced concrete columns shall be taken as:

(*a*) In flat slab construction the clear distance between the floor and under side of the capital;

(*b*) In beam-and-slab construction, the clear distance between the floor and the under side of the shallowest beam framing into the column at the next higher floor level;

(*c*) In floor construction with beams in one direction only, the clear distance between floor slabs;

(*d*) In columns supported laterally by struts or beams only, the clear distance between consecutive pairs (or groups) of struts or beams, provided that to be considered an adequate support, two such struts or beams shall meet the column at approximately the same level and the angle between the two planes formed by the axis of the column and the axis of each strut respectively is not less than 75 deg. nor more than 105 deg.

When haunches are used at the junction of beams or struts with colums, the clear distance between supports may be considered as reduced by two-thirds of the depth of the haunch.

The radius of gyration of a column shall be computed from the concrete area of the core and the transformed section of the longitudinal steel area, that is, the actual area of the steel multiplied by n, this assumed to be distributed uniformly around the periphery of the core.

L-3: *Design of Spiral Columns.*—The safe axial load on columns reinforced with longitudinal bars and closely spaced spirals enclosing a circular core, shall not be greater than that determined by Formula (22).

$$P = A\ [l + (n - l)p]\ [300 + (0.10 + 4p)f'_c] \qquad (22)$$

where P = total safe axial load on column in which h/R (the unsupported length divided by the radius of gyration) is less than 40;

A = area of the concrete core enclosed within the spiral; the diameter of the core (or of the spiral) shall be taken as the distance center to center of the spiral wire;

p = ratio of effective area of longitudinal reinforcement to area of the concrete core;

$f'c$ = ultimate strength of concrete at 28 days as defined in Sec. G-3.

The longitudinal reinforcement shall consist of at least six bars of minimum diameter of ½ in., and its effective cross-sectional area shall not be less than 1 per cent nor more than 6 per cent of that of the core.

The spiral reinforcement shall be not less than one-fourth the volume of the longitudinal reinforcement. It shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. The spacing of the spirals shall be not greater than one-sixth of the diameter of the core and in no case more than 3 in.

Reinforcement shall be protected everywhere by a covering of concrete cast monolithic with the core, which shall have a minimum thickness of 1½ in.

L-4: *Design of Columns with Lateral Ties.*—The safe axial load on columns reinforced with longitudinal bars and separate lateral ties shall be not greater than that determined by Formula (23).

$$P = (A_c' + A_s n) f_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

where A'_c = net area of concrete in the column (total column area minus area of reinforcement);

A_s = effective cross-sectional area of longitudinal reinforcement; and

f_c = permissible compressive stress in concrete and shall not exceed $0.20f'$.

The amount of longitudinal reinforcement shall not be less than 0.5 per cent nor shall the amount considered in the calculations be more than 2 per cent of the total area of the column. The longitudinal reinforcement shall consist of not less than four bars of minimum diameter of ½ in., placed with clear distance from the face of the column not less than 2 in.

Lateral ties shall be at least ¼ in. in diameter spaced not more than 8 in. apart.

L-5: *Bending in Columns.*—The bending moments in interior and exterior columns shall be determined on the basis of loading conditions and end restraint, and shall be provided for in the design.

In flat slab construction, a bending moment at all interior columns equal to $W_1 l/40$ shall be assumed to cover all ordinary cases of unequal

loading in floors and roofs; where W_1 = the total live-load on one panel and l = the span center to center of columns lengthwise of the panels. For known eccentric loads or uneven spacing of columns, computation of moments shall be made accordingly. Resistance to these bending moments shall be provided in the columns immediately above and below in direct proportion to the values of their ratios of I/h (See Sec. H-6 and L-2). In columns supporting roofs the moment shall be resisted by the column below.

Wall columns in flat slab construction shall be designed to resist bending in the same manner as interior columns except that the total live- and dead-load in the panel shall be used instead of the live-load only when computing the moment. Any counter moment due to the weight of the structure that projects beyond the column center line may be deducted from the moment computed as just described.

The recognized methods shall be followed in calculating the stresses due to combined axial load and bending. The limiting unit stresses shall be as follows:

(a) **With Spiral Reinforcement**—The compressive unit stress at the extreme fibre on the concrete within the core area under combined axial load and bending shall not exceed the value given by the expression $300 + (0.10 + 4p)f'c$.

(b) **With Lateral Ties**—Additional longitudinal reinforcement may be used if required to provide for the bending stresses, and the compressive unit stress at the extreme fibre on the concrete under combined axial load and bending may be increased to $0.30f'_c$. The column section, however, shall not be less than that required by the provisions of Sec. L-4 where axial load alone is considered. The total amount of reinforcement considered in the computations shall be not more than 4 per cent of the total area of the column.

Tension in the longitudinal reinforcement due to bending of the column shall not exceed 16,000 lb. per sq. in.

L-6: *Composite Columns.*—The safe load on composite columns in which a structural steel or cast-iron column is thoroughly encased in a circumferentially reinforced concrete core shall be based on a certain unit stress for the steel or cast-iron core plus a unit stress of $0.25f'_c$ on the area within the spiral core.

The unit compressive stress on the steel section shall be not greater than that determined by Formula 24.

$$f_r = 18{,}000 - 70\ h/R \qquad (24)$$

but shall not exceed 16,000 lb. per sq. in.

The unit stress on the cast-iron section shall be not greater than that determined by Formula 25.

$$f_r = 12{,}000 - 60\ h/R \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

but shall not exceed 10,000 lb. per sq. in.

In Formulas 24 and 25,

f_r = compressive unit stress in metal core, and

R = least radius of gyration of the steel or cast-iron section.

The diameter of the cast-iron section shall not exceed one-half of the diameter of the core within the spiral. The spiral reinforcement shall be not less than 0.5 per cent of the volume of the core within the spiral and shall conform in quality, spacing and other requirements to the provisions for spirals in Sec. L-3.

Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders. The area of the concrete between the spiral and the metal core shall be not less than that required to carry the total floor load of the story above on the basis of a stress in the concrete of $0.35f'_c$, unless special brackets are arranged on the metal core to receive directly the beam or slab load.

L-7: *Structural Steel Columns.*—The safe load on a structural steel column of a section which fully encases an area of concrete, and which is protected by an outside shell of concrete at least 3 in. thick, shall be computed in the same manner as for composite columns in Sec. L-6, allowing $0.25f'_c$ on the area of the concrete enclosed by the steel section. The outside shell shall be reinforced by wire mesh weighing not less than 0.2 lb. per sq. ft. or by ties or spirals of equal weight, and with a maximum spacing of 6 in. between strands or hoops. Special brackets shall be used to receive the entire floor load at each story. The safe load in steel columns calculated by Formula 24 shall not exceed 16,000 lb. per sq. in.

L-8: *Long Columns.*—The permissible working load on the core in axially loaded columns which have a length greater than 40 times the least radius of gyration of the column core ($40R$) shall be not greater than that determined by Formula 26.

$$\frac{P'}{P} = 1.33 - \frac{h}{120\ R} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

where P' = total safe axial load on long columns;

P = total safe axial load on column of the same section whose h/R is less than 40, determined as in Sec. L-3 and L-4; and

R = least radius of gyration of column core as defined in Sec. L-2.

CHAPTER M.

FOOTINGS.

M-1: *General.*—The requirements for flexure, shear and bond of Chapters H, I and J shall govern the design of footings except as hereinafter provided.

M-2: *Loads.*—Footings resting directly on soil or on piles shall be proportioned as to area or number of piles on the basis of the total column load plus the weight of the footing itself. For computations of moments and shears, an upward reaction per unit area or per pile shall be based on the total column load (not including the weight of the footing itself) divided by the area or by the number of piles.

M-3: *Sloped or Stepped Footings.*—Footings in which the thickness has been determined by the requirements for shear as specified in Sec. I-7, may be sloped or stepped between the critical section and the edge of the footing, provided that the shear on no section outside the critical section exceeds the value specified, and provided further that the thickness of the footing above the reinforcement at the edge shall not be less than 6 in. for footings on soil nor less than 12 in. for footings on piles. Sloped or stepped footings shall be cast as a unit.

M-4: *Bending in Footings.*—The critical section for bending in a concrete footing which supports a concrete column or pedestal, shall be considered to be at the face of the column or pedestal. Where steel or cast-iron column bases are used, the moment in the footing shall be computed at the middle and at the edge of the base; the load shall be considered as uniformly distributed over the column or pedestal base.

The bending moment at the critical section in a square footing supporting a concentric square column, shall be computed from the load on the trapezoid bounded by one face of the column, the corresponding outside edge of the footing, and the portions of the two diagonals. The load on the two corner triangles of this trapezoid shall be considered as applied at a distance from the face equal to six-tenths of the projection of the footing from the face of the column. The load on the rectangular portion of the trapezoid shall be considered as applied at its center of gravity. The bending moment is expressed by Formula 27.

$$M = \frac{w}{2} (a + 1.2c) c^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

where M = bending moment at critical section of footing;
a = width of face of column or pedestal;
c = projection of footing from face of column; and
w = upward reaction per unit of area of base of footing.

For a round or octagonal column, the distance a shall be taken as equal to the side of a square of an area equal to the area enclosed within the perimeter of the column.

The reinforcement in each direction in the footing shall be determined as for a reinforced concrete beam with the limiting stresses specified in Sec. G-3; the effective depth shall be the distance from the top of the footing to the plane of the reinforcement. The sectional area of reinforcement shall be distributed uniformly across the footing unless the width is greater than the side of the column or pedestal plus twice the effective depth of the footing, in which case the width over which the reinforcement is spread may be increased to include one-half the remaining width of the footing. In order that no considerable area of the footing shall remain unreinforced, additional reinforcement shall be placed outside of the width specified, but such reinforcement shall not be considered as effective in resisting the calculated bending moment. For the extra reinforcement a spacing double that within the effective belt may be used.

The extreme fiber stress in compression in the concrete shall be kept within the limits specified in Sec. G-3. The extreme fiber stress in sloped or stepped footings shall be based on the exact shape of the section for a width not greater than that assumed effective for reinforcement.

M-5: *Footings Other than Square.*—A rectangular or irregularly shaped footing shall be computed by dividing it into rectangles or trapezoids tributary to the sides of the column, using the distance to the center of gravity of the area as the moment arm of the upward forces. Outstanding portions of combined footings shall be treated in the same manner. Other portions of combined footings shall be designed as beams or slabs.

M-6: *Shearing and Bond Stresses.*—See Sec. I-7 and J-1 to 5 inclusive.

M-7: *Transfer of Stress at Base of Column.*—The compressive stress in longitudinal reinforcement at the base of a column shall be transferred to the pedestal or footing by either dowels or distributing bases. When dowels are used, there shall be at least one for each column bar, and the total sectional area of the dowels shall be not less than the sectional area of the longitudinal reinforcement in the column. The dowels shall extend into the column and into the pedestal or footing not less than 50 diameters of the dowel bars for plain bars, or 40 diameters for deformed bars.

When metal distributing bases are used, they shall have sufficient area and thickness to transmit safely the load from the longitudinal reinforcement in compression and bending. The permissible compressive unit stress on top of the pedestal or footing directly under the column shall be not greater than that determined by Formula 28.

$$r_a = 0.25 f'_c \sqrt[3]{\frac{A}{A^1}} \qquad (28)$$

where r_a = permissible working stress over the loaded area;

A = total area at the top of the pedestal or footing;

A' = loaded area at the column base;

f'_c = ultimate compressive strength of concrete. (See Sec. D-1).

In sloped or stepped footings A may be taken as the area of the top horizontal surface of the footing or as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the loaded area A', and having side slopes of 1 vertical to 2 horizontal.

M-8: *Pedestals without Reinforcement.*—The allowable compressive unit stress on the gross area of a concentrically loaded pedestal or on the minimum area of a pedestal footing shall not exceed $0.25f'_c$, unless reinforcement is provided and the member designed as a reinforced-concrete column.

The depth of a pedestal or pedestal footing shall be not greater than three times its least width and the projection on any side from the face of the supported member shall be not greater than one-half the depth. The depth of a pedestal whose sides are sloped or stepped shall not exceed three times the least width or diameter of the section midway between the top and bottom. A pedestal footing supported directly on piles shall have a mat of reinforcing bars having a cross-sectional area of not less than 0.20 sq. in. per foot in each direction, placed 3 in. above the top of the piles.

DISCUSSION.

F. R. McMillan.—This committee was commissioned by the Board of Direction of the Concrete Institute to prepare building regulations based on the recommendations of the Joint Committee. We were instructed to follow these wherever, in our opinion, they were suitable for a building code, and to make only such modifications as in the opinion of the committee should be made in the interest of proper design and construction. Certain modifications were, of course, necessary, for the Joint Committee report is in the form of a specification, and as such covers in considerable detail, methods of design and instructions for carrying out the work. In a building code it is desirable to set up only those broad limits within which proper design and construction can be carried out, leaving a wide latitude for further improvements in the art or changes in materials and methods. Certain clauses have been added pertaining to the application for building permits, changes of plans, load tests on completed structures, etc. There are clauses necessary to a building code but naturally not included in a specification. Mr. McMillan.

We have found it desirable to make a good many minor changes from the wording of the Joint Committee report, partially with the idea of condensing the thought, eliminating clauses that do not apply, and partially to put it in what might be called building code language. In addition to these minor editorial changes, we have made a number of changes of considerable importance. We have included air-cooled blast-furnace slag as one of the permissible aggregates. We have combined the fine and coarse aggregates into one paragraph in specifying quality, and have stipulated percentage limits for the ordinary deleterious materials which are usually covered by the specification "shall be free from injurious amounts." We have also eliminated from the specification of quality of aggregate, any reference to grading. In the minds of the committee, grading is a feature of the proportioning of concrete and not of the quality of the aggregate.

Another important departure from the recommendation of the Joint Committee has to do with metal reinforcement. We have eliminated the hard-grade billet steel and rail steel for all bars larger than ¾ in. where bending is required. In the matter of the quality of concrete we have made perhaps the greatest departure from the report of the Joint Committee. We have based the quality of concrete on the water-cement ratio, permitting a rather wide latitude in aggregate proportions but requiring the concrete to puddle readily into the forms.

C. A. P. Turner.—The construction of a beam or slab in which the depth is small relative to the span length requires certain precautions if the work is to keep its shape. In the cool weather of the spring and fall Mr. Turner.

Mr. Turner. as also in the winter, the better way is to heat the water with which the concrete is mixed to 100 or 120 deg. so that the setting may proceed rapidly and the uncertainty of complete curing when the forms are removed may be avoided. Concrete which is chilled in the early stages of curing may, after the removal of the forms, sweat during a rise of temperature and so soften as to get out of shape, and although in most cases it ultimately hardens again so that it can stand a good test load, it looks badly, may be cracked and its strength questioned, notwithstanding its load carrying capacity. Curing of concrete that has been chilled in its initial stages is slowly accounting for some failures. Heating the water to 100 deg. F. lessens the surface tension, results in a stronger concrete and avoids the uncertainty noted and the possibility of the work getting out of shape.

Joint Committee rules on columns have been apparently devised with little regard to the variation of the elements which determine strength. Vertical reinforcement up to one or two per cent adds little to the strength of the concrete shaft having a diameter one-sixth or one-eighth the length. Spiral hooping without vertical steel toughens the shaft but under test it commences to crack and scale on the outside at approximately the ultimate compressive resistance of the unreinforced shaft of the same grade of concrete. When the reinforcement consists of a combination of hooping with the proper proportion of vertical steel, scaling and checking does not occur before 80 or 90 per cent of the ultimate strength is developed even when ordinary ties were used and the vertical steel distributed in the outer portion of the column a large increase in the strength is secured over the plain concrete or the strength of a prism which is vertically reinforced without ties. For example, in Bach's tests 1¼ per cent of vertical steel with ¼-in. ties spaced 2½ in. c. to c. resulted in greater ultimate strength than 4.6 per cent vertical steel with 10-in. tie spacing. An increase of four-fold the amount of vertical steel increased the strength a smaller amount than an increase of four times in the volume of the small wire ties. The Joint Committee, failing to appreciate this relation, would permit ¼-in. diameter ties for a 30-in. column, which would correspond relatively to the cross-section of hay wire in the practice of the contractor who put up the Cleveland building some years ago six or seven stories high and had a complete collapse before he finished. The committee rules present a dangerous oversight in carefully providing for fireproofing for the hooped column, which is the most reliable and permitting the entire section of the square rodded column to be counted upon as net section. The dangerous character of these rules has been demonstrated again and again by the extensive damage done to the rodded column by heat in fires as compared to the toughness and resistance of the hooped and vertically reinforced type. Notwithstanding such experience the committee permits higher values for the most dangerous type than they do for the more conservative and safer type when these sections are small. This is brought about in part by the inconsistency with respect to fireproofing noted. The

relative proportion of the ties and the vertical steel in the rodded column and of the vertical steel and the hooping in the latter type have been given scant consideration by the committee since by increasing the amount of hooping much higher working stresses may be used with an increase in safety over the formula advocated by the committee. They present in their report no experimental evidence as to the effect of the variation in the relative percentages of the vertical steel ties and hooping. They have selected unfortunately those percentages which are uneconomical and fail to develop the maximum strength for the material used. Mr. Turner.

The Joint Committee report was a distinct disappointment in its unscientific treatment of shear. The Concrete Institute appointed a committee some years ago that drew up a set of rules (empirical it is true) for the shear resistance of beams and the like. They permitted different values for what they termed punching shear and ordinary shear, treating as punching the resistance of the restrained or continuous beam, and as ordinary shear the resistance of the simply supported beam. If we consider the divergent types of rotational strain under flexure we see that an original square on the face of a rectangular beam is deformed by bending into a trapezoid at the point of maximum curvature. Both diagonals of the square have been shortened in the compression zone and both have been lengthened in the tension zone. There is no rhombic distortion where the moment is a maximum. This latter kind of shear strain increases from the center toward the end in the simply supported beam and from the center to the point of inflection in the restrained beam decreasing from the point of inflection to the support and the total rotational strain is the sum of these two components. The rhombic component reduces to zero in the restrained beam at the support leaving only the trapezoidal component. Whereas at the end it is a maximum in the simply supported beam. Accordingly the ground work for the distinction between the so-called punching shear and shear may be roughly established. But each rotational deformation results in cracking or failure of the material when it exceeds a definite magnitude. Because these combined deformations are readily computed and readily combined they should be treated in a rational manner. Rules of thumb and guess work should be eliminated from the rules of design. The committee has made no advance whatever in this respect but have complicated their specification with numerous rules which have no scientific foundation.

The importance from the economic standpoint of a correct grasp of the nature of shear distortion or rotational strain lies in the fact that for certain ratios of depth to span length the resistance of the restrained beam to external shear force may be four or five times as great in terms of unit stress per square inches of cross-section as the shearing resistance of the simply supported beam and hence scientific analysis warrants economy in materials and a material saving without sacrifice of safety. The extremes to which lack of familiarity with the fundamental principles governing the resistance of rotational strain is exemplified in the com-

Mr. Turner. mittee's report on plates. The fundamental error in the ordinary building code rules as well as in those of the Joint Committee may be illustrated by experiment almost kindergarten in its simplicity. If we take a square plate of celluloid, drill it in the center, support it on four side and suspend a weight on a string through the hole, measure the deflection, then remove two of the supports and measure the deflection of the plate under the same load when supported on two opposite sides, the deflection so found will be more than four times as great when the plate is supported on two sides than the measured deflection when it is supported on four sides. Now if the resistance of the plate were merely that of bending in two directions at right angles the effective moment would be divided by two because of four supports in place of two and the deflection should be halved instead of quartered. But the plate is not only bent but twisted and by the twisting the load concentrated at mid-span is distributed along the sides with its greatest intensity at about the quarter-point of the sides. Now the distribution of the reaction outward along the sides from the center of the sides at right angles to the adjacent supports represents a force which is virtually a negative moment as opposed to the positive moment due to the load. Hence from the distribution of the reaction the magnitude of this virtual negative moment is more than half the positive moment due to the load. And whereas the effective-bending moment on the plate when supported on two opposite sides is $WL/4$, it is less than $WL/16$ when supported on four sides or were the load uniformly distributed the effective moment by the same reasoning would be less than $WL/24$ in each direction instead of $WL/12$ as ordinarily figured. In like manner an interior panel of a column supported plate if reinforced so that mat action is efficient will be called upon to resist not more than $WL/24$ distributed between mid-span and supports in the diagonal direction and in the direction of the column lines as well in the square panel. The fundamental error of the committee's report lies in their erroneous endeavor to determine the moment from three planes of zero shear instead of considering the eight different planes by which the panel is divided, to wit, the planes of zero shear which follow the column lines four in number, median lines parallel thereto two in number, and the diagonal lines two in number. When these various triangles are considered it is observed that for uniform load one-fourth of the panel load is transferred in a diagonal direction to each column. The moment of the load on these squares would be $WL/16$ total in the diagonal direction resisted partly over the supports and partly between the supports but as opposing this there is a twisting moment which sustains the half squares of the load which are transformed to the opposite columns in amount equaling $\frac{1}{4}W$ times one-twelfth the diagonal span or $WL/48$ which deducted from $WL/16$ leaves a total applied moment in a diagonal direction of $WL/24$, assuming the columns as mere points of support.

The computation of effective moment which is in proportion to deflection is an exceedingly simple matter not understood at all by our com-

mittee and because they do not understand these elementary relations their measured stresses in flat slabs fail to agree with computed stresses according to their theory within 400 or 500 per cent. It would seem as though flat-plate construction had been on the market long enough so that guess work might be eliminated from the committee's rules and rational theory replace the guess work and empiricism. The writer has developed the theory of plates in Sec. III, "Elasticity and Strength of Materials," in such wise that the stress mechanism curves of displacement and equipotential are fully developed. He has also presented a like analysis of beams and shear distortion in Sec. II both for homogeneous and concrete beams. Mr. Turner.

The Joint Committee explanation for the divergence between measured stress and computed stress on the ground that the tensile resistance of the concrete is responsible therefor is rendered irrational by the fact that the resistance found present when the steel is omitted is less than two per cent of that which they credit the concrete with possessing when it is weakened and its section reduced by steel embedment. Voting on the adoption of rules as is customarily done in our committees without regard to the agreement of these rules with experimental fact retards scientific progress in the industry and tends to perpetuate the myth that theoretical rules and practical knowledge are unrelated and antipodal in character.

PROF. W. K. HATT.—I have just been wondering what all this is about. As I understand it, a building code deals with public safety and all these matters of improved practice and of relative economy have no legal place in a building code. The Building Code Committee of the Department of Commerce, of which Mr. Woolson is chairman, sees a building code as applied to all sorts and conditions of men. A code must be widely applicable. We have tried in our Building Code Committee to tie up the elaborate Joint Committee regulations with a very simple code which can apply to small towns where there is no building official and no technical direction. We tried to do that by putting a clause in, something of this kind: In the absence of rules adopted by the building official and except as otherwise specifically provided in this code, the formulas and specifications of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, representing the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering Association, the American Concrete Institute, and the Portland Cement Association, shall be assumed in calculating the strength of concrete and reinforced-concrete structures, and designs shall conform therto. That ties up a very intricate document with a code that is in very simple language to cover small town men. Prof. Hatt.

Another difficulty in the adoption of a building code is the expense of the required publication, say, three times in three papers. Therefore it should be reduced to the simplest terms. Matters of good practice and information can be covered in a supplementary handbook which will be effective, even if not in the code itself. How far is it necessary to put

Prof. Hatt. into a building code all these matters of relative economy and competition in design when a building code is primarily the exercise of police power to govern public safety?

Mr. McMillan. F. R. McMILLAN.—The committee holds the theory that a building code should avoid detailed specifications of methods or materials, but should define the broad limits within which safe construction can be assured, leaving the way open as much as possible for improvements in the art. However, custom has decreed that certain items be included as a guide to practice, such as methods of computations, customary limits of sizes, etc., and as a concession to this custom, the committee has included in its regulations, items that do not belong there under a strict interpretation of the province of a building code.

Mr. Cross. HARDY CROSS.—I am very strongly of the opinion that the Joint Committee report needs very much more full discussion by the profession before it is accepted by it or before any code is based upon it. I do not believe that a great deal of the Joint Committee's report is warranted by any public data or by any clear analysis that has been published. The Shipping Board's tests in regard to shear have not been published. I do not think the profession should be requested to accept a report based on arbuments which are not stated and data which are not published. The old Joint Committee report was very conservative in some respects, perhaps too conservative, but in breadth of view I believe it is a very remarkable document as compared with the present report. It was less mandatory and less dictatory. It placed emphasis on the essentials and it did not make the method of figuring designs mandatory, and showed few hoofprints of hobby horses in the concrete floor. The present report is incorrect in many places. I believe it is incorrect in Sec. 118, where it is stated that the flanges of beams do not assist in carrying the shear. I do not think that is the intention of the committee, yet that is plainly stated, and Sec. 122 is ambiguous. The same may be said of Sec. 123-B, 182 and many others. I have heard many different interpretations of several of these clauses made by men who were certainly in a position to interpret them intelligently. Some of the clauses of the present Joint Committee report are simply ambiguous. The present Joint Committee, it seems to me, vacillates between dogmatism in certain non-essentials, as for example, in retaining walls, where they have specified that the cantilever of a retaining wall shall be treated as a cantilever; it could not be treated as anything else. They have specified also that the counterfort shall be specified as rectanglar beams, which they are not. They pass to extreme vagueness in certain essentials and specify how we shall handle irregular beams, columns and slabs in all irregular cases. It is, I believe, radical and unprecedented as regards columns and especially as regards steel columns. It seems to me that the report of the Joint Committee makes concrete a poor thing of rules and formulas and a sort of red-headed stepchild in the structural engineer's office. I do not believe that its code is helping the development of concrete construction. It needs to be clearer

and a great deal simpler. It should remove from the art the shackles of a detailer's perspective. I wonder how many men here whose duty has not required it, have discriminately read the report? It breaks radically in half a dozen ways from American practice and precedent. Mr. Cross.

Mr. Mylrea.

T. D. MYLREA.—The progress report of Committee E 1 not being available in printed form makes it rather difficult to discuss. However, since the chairman of the committee has stated that this report is based upon the Joint Committee Specifications for Concrete and Reinforced Concrete, the writer will refer to specific clauses in the Joint Committee report, in order that Committee E 1 may have definite clauses before them in considering his discussion.

In Clause 104, Sec. b, Formulas 6, 7, 8, and 9 neglect the fact that compression steel and concrete cannot simultaneously occupy the same space. This is not serious in Formulas 6, 7, and 8, except that their inclusion at all implies a desire for exactness. In Formula 9, however, this difference may cause a considerable variation in the computed unit compressive stress in the concrete.

In Clauses 115 and 118 it might be possible to make the distinction between flange and stem a little clearer. Clause 115, for example, would imply that the flange is composed only of the overhanging portions of a T-beam, while in the footnote "taking into account the compression in the stem" the implication is that all that portion above the lower surface of the T, and extending clear across the beam, constitutes the flange.

In Clause 121, Formula 29 applies only to beams in which the compression face is parallel to the tensile reinforcement.

Formula 30, in Clause 125, might have been so stated as to give a clue to its derivation. As it stands it is irrational in form, and therefore its origin should be made evident before it can be generally adopted.

In Clauses 120 to 130 it has been made impossible to design beams without web reinforcement, unless in Formulas 31 and 32 "s" be taken as infinity. There are cases where it might be quite desirable to design beams without web reinforcement. Formula 31 does not seem necessary; Formula 32 being correct for all angles, when the assumption is made that the diagonal compression always acts at 45°.

It is not right to increase the value of the concrete in shear as would be permitted by Clause 128, simply because the reinforcement is anchored, since shear reinforcement merely prevents the opening of cracks and not their formation.

Clauses 129 may lead to rather peculiar results. In a short beam it may be impossible to bend reinforcement except in a single plane. In a continuous girder, for example, carrying concentrated loads from beams at the third points of an 18-ft. span, panels square, floor load say 500 lb. per sq. ft., it is almost imperative to make all bends in one plane at 45 deg. In this case, not uncommon, a limit of 75 lb. per sq. in. is needlessly conservative. As a whole the clauses relating to shear in beams seem far less applicable to continuous beams than to simple beams.

Mr. Mylrea. Formula 34 also applies only to beams in which the compression face is parallel to the tensile reinforcement. For example, in a cantilever beam, rectangular in plan, triangular in elevation, horizontal along the top edge, with a concentrated load at the point, the steel stress is constant for its whole length. In such a case, since there is no variation of the steel stress, no bond is developed, and full anchorage is required at both ends, regardless of the value of V. If the same beam were loaded uniformly, the bond stress would be constant from end to end. This is the case in sloped top footings, and indicates that there may be very considerable bond even to the very ends of the rods, although from Formula 34 the bond stress there should be zero. Footings being probably the most important part of a building, this item is of grave importance. Of course these remarks hold true for all tapered beams such as counterforts, buttresses and balcony girders.

In Clause 137, Formula 35 can easily lead to absurd results, since the length x includes the hooks. This may mean that a hook embedded only a short distance in a supporting member would be allowed to carry a heavy stress. Obviously it is possible for such a hook to dislodge the piece of concrete between it and the surface. In ordinary construction work hooks are of much less value than is usually supposed, since in anchoring a rod they also exert a very serious splitting force. The problem of shear and bond is far from settled yet, and while some of the governing conditions have been recognized in this report there is still a great deal to be said upon the subject.

Formula 37, in Clause 145, giving the required thickness of flat slabs, is at least three times too long. It implies that we have an accurate knowledge with regard to flat slabs which does not exist, and since the whole design is largely empirical a shorter and more easily applied formula would serve the purpose just as well as Formula 37.

In Clause 156, Formulas 40 and 41, there appears in the denominator the following expression: $.067\sqrt[3]{p\,n}$ There seems no reasonable justification for this term in either formula. According to the footnote applying to Formula 3, Clause 104, this term is approximately equal to our familiar friends jk. Since it would take much longer to solve Formulas 40 and 41 as stated than it would if jk had been used, there seems no reasonable justification for the introduction of an approximate value for one theoretically exact.

There appears to be no reason for either Formula 40 or 41, in the form given, since both of them merely give the stress resulting from the application of the usual laws of statics without reduction for flat slabs. If the reduction is permissible, as implied in Formula 39 for the steel stresses, why not in Formulas 40 and 41 for the concrete stresses? Flat slabs seem to be no more unsafe as regards concrete stresses than for steel stresses.

The regulations regarding columns are loose in many respects. There seems to be no reason, for example, why it is less safe to use vertical rods

thoroughly tied together than to use a structural steel core composed of, say, four angles lightly laced together; yet we are limited to two per cent on vertical rods without spirals, while no limit is placed on the reinforcement if in the form of structural steel. It would appear that longitudinal reinforcing without spirals is discountenanced by the Joint Committee. While certainly unsafe if buckling of the bars is not prevented by thorough tying, the low limit is extreme. Mr. Mylrea.

In the case of composite columns, Clause 168, some limit should be placed on the l/r of the steel core, the only limit imposed being that resulting from the applications of Formulas 45 and 46.

In Clause 170 it is possible with the reduction formula given in Formula 47 to design a reinforced concrete column 1 ft. sq. with 2 per cent of steel in it, 47 ft. long. Even with a length of 40 ft. such a column would carry at the top a very considerable load.

Concrete Products Plant Operation.

Submitted by Committee P-6.

During the past three years the Committee on Concrete Products Plant Operation has submitted reports on

(1) Design of plants (1922).
(2) Cost Keeping (1923).
(3) Economical Manufacturing Processes (1924).

The committee felt that for 1925 it should produce,

(A) A simplified recommended practice for manufacturing concrete masonry units, easily understood and applied by products manufacturers.
(B) A practical application of the data submitted in the 1924 report on the manufacture of light concrete building tile.
(C) Investigation of the effect of various methods of curing.

In order to facilitate the work of the committee, sub-committees were appointed to handle each subject. The appendices to this report contain the simplified Recommended Practice and also detailed data developed on the manufacture of light concrete building tile and on curing. These data have been studied by the committee and the following recommendations are offered on,

(A)—Manufacture of Light Concrete Building Tile.

(1) Concrete building tile complying with the requirements of the American Concrete Institute can generally be produced more economically than by present plant methods if accurate control is secured of the quality and grading of aggregates, quantities of water, cement and aggregate, and method of curing.

(2) Concrete should be mixed as long as possible. If it is mixed 6 minutes there is no apparent advantage in mixing dry part of this time.

(3) With the amount of mixing water giving the best workability, there is no disadvantage in allowing concrete to remain unused up to two hours if the concrete is covered so that no evaporation takes place.

(4) The maximum amount of water producing concrete that will not bulge or slump when removed from the machine, gives tile of maximum strength.

(B) Method of Curing.

(1) The most economical condition of curing is in an inclosed room at 70 deg. F. for 24 hours followed by outdoor storage until the product

has reached the required strength. The product should be sprinkled during the first 7 days of outdoor storage when the temperature is not below 45 deg. F.

(2) The proportion of cement to aggregate should be such that the required strength is obtained with storage for 28 days at temperatures above 70 deg. F. With temperatures below 70 deg. F., the required strength can be obtained, at 28 days by increasing the proportion of cement to aggregate or by lengthening the storage period.

(3) Where it is desirable to use products at early ages, the strength can be increased by adding cement. In general for each 1 per cent in cement there is 1 per cent gain in strength desired.

(4) With outdoor temperatures above 70 deg. F. it is not of advantage to inject heat or moisture into the curing rooms. It is recommended, however, that the walls and floor be soaked with water just prior to filling the room to minimize evaporation of water from the block.

(5) When curing rooms are being heated during cold weather, a small amount of moisture should be injected into the rooms while the temperature in the rooms is rising.

(6) Products may be stored outdoors at 0 deg. F. after one day's curing in moist air at 100 deg. F.

(7) To minimize exposure and evaporation when stored in the yard, products should be piled compactly in large piles with cores vertical and a top layer with cores horizontal.

The committee wishes to thank the Humboldt Gravel & Tile Co., Humboldt, Iowa; Bert Carey & Co., Forest Park, Ill.; Structural Materials Research Laboratory, Lewis Institute, Chicago; and the Buffington laboratory of the Universal Portland Cement Co. for their co-operation in carrying out these tests. The committee wishes to express its appreciation to the members of the subcommittees for the work done by them and also to Messrs. M. C. Tobias and Benjamin Wilk of the Universal Portland Cement Co., and A. Timms of the Structural Materials Research Laboratory, who are not members of the committee, for their work in the making of tests and in the preparation of the reports.

J. W. LOWELL, *Chairman*
C. L. BOURNE, *Secretary.*

SUBCOMMITTEE ON RECOMMENDED PRACTICE FOR MANUFACTURING BUILDING BLOCK, TILE AND BRICK.

C. L. Bourne, *Chairman*
E. W. Dienhart
Austin Crabbs
W. D. M. Allan.

SUBCOMMITTEE ON CONCRETE BUILDING TILE.

Benjamin Wilk, *Chairman* (Acting for J. W. Lowell)
C. L. Douthett
H. F. Gonnerman.

SUBCOMMITTEE ON CURING.

H. F. Gonnerman, *Chairman*
Bert Carey
E. W. Hilker
Lawrence Cox
C. L. Bourne.

Recommended Practice for the Manufacture of Concrete Building Block, Building Tile and Brick.

Submitted by Committee P-6.

I. FOREWORD.

These recommendations are intended to assist in the manufacture of concrete building block, building tile and brick that will meet the requirements of the Standard Specifications of the American Concrete Institute. Particular attention has been given to those features which will help to produce quality products economically.

II. MATERIALS.

(A) Cement.

1. Cement shall meet the requirements of the current standard specifications for portland cement of the American Society for Testing Materials. Cement shall be stored so that it will not be exposed to moisture. **Portland Cement.**

Notes—Cement may be stored without loss of quality for several months if it is kept away from moisture. The storage room should be so constructed as to eliminate drafts of air through cracks, windows and doors. The height of the room should not be much greater than the height to which the cement is to be piled. If a wooden floor is used, a double floor with waterproof paper between is recommended.

When cement is stored for a considerable time or in high piles there is a tendency for it to assume a condition known as "warehouse pack." This condition is not harmful as the "warehouse pack" will be completely broken up during the mixing of the concrete. "Warehouse pack" can also be broken up completely by rolling the sack. Where cement is piled over 7 sacks high it is advisable to pile the outside sacks in alternate headers and stretchers to eliminate the possibility of piles over-turning.

One complete shipment of cement should be used before any later shipment is used.

The use of bulk cement has been found economical in products plants. When bulk cement is used, it is most accurately measured by weight.

Ample storage space should be provided to insure that the plant will not be shut down due to delayed shipments.

(B) Aggregate.

2. Aggregate should consist of sand, pebbles, crushed stone, crushed blast furnace slag, or other approved inert materials with similar character- **General.**

istics or a combination thereof, having strong durable grains free from injurious amounts of dust, lumps, soft or flaky particles or shale, and should not contain injurious amounts of vegetable or other organic matter as determined by the colorimetric test. The diameter of the largest particle should not be greater than one-half of the thickness of the thinnest wall of the concrete building unit in which it is used.

In no case should aggregates containing frost or lumps of frozen material be used.

Notes—The grading of the aggregate particles has an important bearing upon the strength of the concrete. This subject is discussed under "Proportions for Mixtures."

Fine Aggregate. Fine aggregate should be considered as the aggregate passing a screen having four (4) meshes to the linear inch. Coarse aggregate should be considered as the aggregate retained on a screen having four (4) meshes to the linear inch.

Injurious amounts of very fine materials in the fine aggregate, such as silt and clay, can be determined by the following method: Fill a 32-ounce graduated prescription bottle to the 14-ounce mark with the fine aggregate to be tested; add water to the 28-ounce mark; shake vigorously for one (1) minute; allow to settle one hour. If more than one and one-half (1½) ounces of sediment appears above the fine aggregate, the material represented by the sample should be rejected. It is not contended that concrete masonry units can not be made with aggregate containing more fine materials than the 10 per cent indicated by this test and still meet Standard American Concrete Institute Specifications. Such aggregate may require enough additional cement to render its use uneconomical.

Colorimetric Test. The colorimetric test is applied in the field to fine aggregate as follows: Fill a 12-ounce graduated prescription bottle to the 4½-ounce mark with the fine aggregate to be tested. Add a 3 per cent solution of sodium hydroxide (caustic soda, obtainable at any drug store) until the volume of fine aggregate and solution, after shaking amounts to 7 ounces. Shake thoroughly and let stand for twenty-four (24) hours. The sample should then show a practically colorless solution not darker than straw color. If the solution is darker than straw color, the fine aggregate represented by the sample should be rejected unless it has passed a mortar strength test in a properly equipped testing laboratory in which case the aggregate in question should show a strength at least equal to that of standard Ottawa sand. Often fine aggregates which show an unsatisfactory colorimetric test can be thoroughly washed and a subsequent test will show that the objectionable organic material has been removed.

Heating Materials. Plants having stationary aggregate bins can easily place steam coils around the inside of the bins to thaw out aggregates during cold weather. With this arrangement, there is an assurance that no frozen material will be used and, in addition, heated materials will assist in the early hardening of the concrete which is especially to be desired in cold weather.

Where there are no material bins, frozen aggregates or those containing frost can be warmed up to proper temperature by placing over steam coils or on a section of pipe or smoke stack over which a fire has been kindled, or by thrusting a perforated pipe into the pile and blowing live steam through it.

Continuous rotary sand dryers and heaters are made by most manufacturers of elevating and conveying machinery. Such equipment is, however, somewhat expensive.

The grading of the particles in aggregate has such an important bearing upon the strength of concrete, that it is unsatisfactory to use bank-run gravel or crushed-run stone in cases where the ratio of fine to coarse varies. It is always safe to separate the fine and coarse material on a screen having four (4) meshes per linear inch and recombine them in definite proportions. If this is not done, the aggregate should not be used unless analyses are made and deficiencies in grading corrected or compensated for. **Grading.**

Fine and coarse aggregates should be stored in separate bins. When it is necessary to pile aggregates outside, material in the bottom of the pile which has become mixed with dirt should be discarded. **Storing.**

(C) Water.

3. Water should be free from injurious amounts of oils, acids, organic material or other harmful matter. **Water.**

Notes—Water fit for drinking purposes is generally suitable for concrete. If there is any question as to the concrete making quality of available water, its suitability should be determined by tests of representative samples in a competent laboratory.

It is advisable to heat the water during cold weather operation because of its effect in hastening early hardening. The simplest method is by introducing a steam nozzle into a barrel of water. There are a number of efficient water heaters on the market.

(D) Coloring Pigments.

4. Only mineral pigments shall be used. These pigments should be fully guaranteed by their manufacturer to contain no ingredients which will be affected by lime, cement and weather. **Pigments.**

(E) Proportions.

5. The proportions of cement to aggregate shall be such that the products will conform in strength and absorption to the current requirements of the American Concrete Institute for such products or conform with any state or municipal building code under whose jurisdiction the products are used. **Proportioning.**

Notes—Following is a summary of strength and absorption requirements for concrete building units as given in the Tentative Standard Speci-

fications for Concrete Building Block and Concrete Building Tile and the Tentative Standard Specifications for Concrete Brick of the American Concrete Institute.

CONCRETE BUILDING BLOCK AND BUILDING TILE.

	Compressive Strength, lb. per sq. in. of gross cross-sectional area as laid in the wall	
	Average of 3 or more Units	Min. for Individual Unit
Heavy load bearing block or tile	1200	1000
Medium load bearing block or tile	700	600
Non-load bearing block or tile	250	200

CONCRETE BRICK.

	Compressive Strength, lb. per sq. in. of gross cross-sectional area as laid in the wall	
	Average of 5 Specimens	Min. for Individual Unit
All concrete brick	1500	1000

Absorption. Maximum absorption of concrete building block and building tile is 10 per cent of the dry weight of the unit, subject to corrections for light weight aggregate concrete. The maximum absorption for concrete brick is 12 per cent of the dry weight of the brick. The methods of testing are described in the Tentative Standard Specifications.

The only manner in which the physical properties of a concrete masonry unit can be definitely determined is by tests in a properly equipped laboratory. Fortunately such laboratories are to be found in every section of the country.

In general, the coarser the grading of an aggregate, the greater will be the strength of concrete resulting from the same proportions of cement to aggregate. Thus the addition of coarse material to aggregates lacking in it, results either in economy or a stronger concrete, or both.

The coarseness of grading of aggregate is limited by its workability in the process or type of molding machine, for when a mixture becomes harsh working, the limit of increased strength due to coarse grading has been reached. Sometimes the grading which will give concrete of the greatest strength will produce units of too rough a texture to be salable. The size of the maximum aggregate is also limited by the size and shape of the concrete unit.

In order to determine the proper proportions of cement to aggregate to produce concrete masonry units of desired quality, units should be made and tested using different combinations of fine and coarse aggregates with various proportions of cement to mixed aggregates, such as 1 : 4, 1 : 6 and 1 : 8. The information thus obtained will serve as a guide to the most economical combinations of available materials.

It is strongly advised that sieve analyses be made on all combinations of aggregates used in the tests outlined above. A simple measure of the grading of aggregates has been developed called the fineness modulus* which is readily calculated from the sieve test. The sieves used are such that the linear dimension of the clear square opening of each sieve is twice that of the next smaller sieve in the series. The following gives the sieve sizes used and illustrates a typical fineness modulus determination.

FINENESS MODULUS OF A COARSE SAND.

Sieve Size or No.	Dimension of square opening (inches)	Per cent by Weight coarser than each sieve
100	0.0058	97
48	.0116	81
28	.0232	63
14	.046	44
8	.093	25
4	.185	5
⅜ in.	.37	0
¾ in.	.75	0
	Fineness Modulus =	3.15

Sieve Analyses.

The sieve analyses are expressed as percentages coarser than each sieve. For example, 97 per cent of the entire sample is coarser than the 100-mesh sieve, 81 per cent is coarser than the 48-mesh sieve, etc. The fineness modulus is the sum of the percentages in the sieve analysis divided by 100.

Increased strength will result by adding coarse material to aggregates having fineness moduli below those shown in the following table:

Aggregate graded from	Economical Range of Fineness Modulus
0 - No. 8	2.50 to 2.90
0 - No. 4	2.90 to 3.50
0 - ⅜ in.	3.50 to 4.25
0 - ½ in.	4.25 to 4.50
0 - ¾ in.	4.50 to 5.00

Checking Grading

Whatever method is employed to obtain most advantageous grading of aggregates, a set of standard sieves furnishes a positive method of checking the grading. If a quantity of material is delivered which has a lower fineness modulus than that desired, a sieve analysis will indicate changes in proportions to maintain an even quality of product. If, on the other hand, delivered aggregate is too harsh, an analysis will disclose the amount of fine material to be added in order to reduce the fineness modulus to the determined standard. The knowledge that variations in the grading of aggregates will affect the strength of concrete products as much as 100 per cent in extreme cases is of little value unless definite means are taken to

*Refer to Bulletin No. 1 "Design of Concrete Mixtures," Structural Materials Research Laboratory, Lewis Institute, Chicago.

control this factor within reasonable limits. Many cases may be cited where manufacturers have increased the yield in number of units per barrel of cement as much as 50 per cent without loss in quality, simply through careful analysis of available aggregates and the application of the principles briefly outlined herein.

III. MANUFACTURE.

(A) Consistency.

Consistency.

6. Concrete to be used in units made on a tamp or pressure machine should be mixed as wet as practicable, allowing immediate removal from the mold, without sagging or distortion. Concrete to be used in units made by the cast process should be mixed as dry as practicable to completely fill the molds and produce acceptable surfaces.

Notes—Most tamped block are made too dry. Most cast block are made too wet. Under given conditions a definite amount of water is required to produce concrete of maximum strength. Any variation from this amount will reduce the strength. Variations of 50 per cent in the strength of concrete are often observed due solely to changes in water content. It is essential, therefore, that the water used be as carefully controlled as the proportions of cement to aggregate.

Tamped Method.

In the tamped method, products can be made wet enough to draw free water to the surface by trowelling and show water web marks upon removal from machine. There is a natural tendency to use too dry a mix because less care in handling is required. When concrete block are made by pressure the concrete can be made wet enough so that a slight excess of water will be forced from the block when the full pressure is applied.

Much drier mixtures may be used with wet cast block than are generally considered possible. Vibrating machines are helpful in settling the concrete.

(B) Mixing.

Mixing

7. Cement and aggregates should be mixed dry until a homogeneous mass is obtained. The materials should be mixed at least two minutes after the water is added.

Notes—An increase of from 30 per cent to 50 per cent may be expected by lengthening the time of mixing from one to ten minutes, in concrete of the consistencies usually employed in products manufactured. Half of this gain in strength may be obtained by increasing the time of mixing from one to two and one-half minutes. Experiments have developed the fact that there can be a great variation in the speed of the mixer without an appreciable change in strength. Products plants should have sufficient mixer capacity so that concrete can be mixed to conform with the time of mixing recommended. Thorough mixing not only results in increased strength but also in better workability.

(C) Compacting.

8. Machine tampers should be used in the manufacture of tamped units. Vibrating devices are recommended for wet cast products.

Notes—Uniformity of compacting is important in all concrete products manufacture. Machine methods are much preferred over hand methods.

(D) Curing.

9. The products should be cured sufficiently to attain the strength recommended herein at twenty-eight (28) days or when used.

Steam Curing.

The term "steam curing" applies to any method of curing where concrete building units are kept in a warm damp atmosphere during the early hardening period and the heat is supplied by steam in a curing chamber. During the curing period it is desirable that the temperature of the curing chamber should not fall below 100 deg. F. When the outside temperature is never below 50 deg. F. the products should be steam cured at least twenty-four (24) hours and then placed in storage and kept moist for at least four days. When the outside temperature falls below 50 deg. F. the products should be steam cured at least forty-eight (48) hours.

Sprinkler Curing.

If cured by sprinkling, concrete products shall be kept moist for a period of at least five days in an enclosed room where the temperature is not allowed to drop below 50 deg. F. and the average temperature is 65 deg F. or above, after which they shall be placed in storage and kept moist an additional five days. When the outside temperature falls below 50 deg. F. the products shall be kept moist in the enclosed room for a period of ten days.

Notes—The hardening of concrete is due to a chemical action which takes place in the portland cement. Moisture is necessary for this action which stops as soon as concrete is dried out. Heat assists the hardening of cement and the concrete attains the strength more quickly in high than in low temperatures. Curing is the process of keeping concrete wet during the early hardening period until it has attained sufficient strength to be successfully used for the purpose for which it is intended. The addition of artificial heat hastens curing and permits the manufacturer to prepare his products for market in the shortest length of time.

The steam curing recommendations given represent the minimum conditions which have been found in practice to give uniform and satisfactory results, and are not intended to convey the impression that additional curing will not be beneficial.

One method of supplying moisture in steam curing chambers is by introducing free steam into the air, either directly from the heating pipe or through water troughs. In some recent installations, the pipe which furnishes steam for moisture is independent of the radiation coils. Another method which has several advantages, is to furnish moisture through fog sprays from a pipe running the length of the curing chamber directly below the roof. Heat is supplied independently through standard steam heating equipment in which the condensed steam returns to the boiler.

The value of keeping products moist in storage after steam curing is not widely recognized by products manufacturers. This practice should become general. The expense is small. Ordinary garden sprinklers, frequently moved, furnish a simple method of keeping products wet in storage piles. Perhaps the best arrangement is a system of overhead pipes with stationary sprays which can cover the entire storage place. Such a system has been installed at several plants. Sprinkling by hand, although not as efficient as either method mentioned, is much better than no wetting.

Curing by sprinkling is satisfactory but takes longer than by steam. In sections where winters are severe, curing by sprinkling is practically prohibited in cold weather because of the cost of heating a room large enough to hold ten days' production.

(E) Admixtures.

Use of Hydrated Lime.

10. The use of admixtures in general is not recommended.

Notes—The "workability" of lean mixtures is slightly increased and the workability of rich mixtures changed but little with the addition of hydrated lime. Under the same conditions, the addition of like amounts of portland cement produces the same results.

Calcium Chloride.]

Calcium chloride in quantities of not more than 3 per cent by weight of the cement has been used as an admixture to increase the strength of concrete during the early hardening period, thus decreasing the length of time before the release of pallets or molds.

Attention to consistency and thorough mixing often overcomes apparent difficulties in concrete mixtures for products. The introduction of another material to the mix adds another variable. Usually it is better to properly manipulate the materials now employed than to introduce new ones.

(F) Surface Finish.

11. Surface finishes shall be composed of suitable materials so applied as to insure permanency of color and texture.

Notes—In general, the indiscriminate use of rock face block, such as in entire buildings, should be discouraged.

Selected Graded Aggregates.

Beautiful and durable surface effects are obtained through the use of selected colored and graded aggregates. Mixtures for surface facings may be determined by trial. Gradings of aggregates having the least voids are best as more aggregate surface will be exposed. Fine material in special surfaces should be avoided because of its tendency to cause surface crazing. Aggregates chosen for surface colorings should stand up upon exposure to weather, thus some marbles, feldspars, sandstones, etc., are to be avoided.

Vapor Spray.

The aggregates may be exposed by removal of the surface film of cement by three general methods depending upon the hardness of the concrete. (1) A very fine gentle spray blown on the surface of the concrete soon after molding, (2) Scrubbing with a stiff fiber or wire brush and

water before the concrete has become too hard, (3) Etching with muriatic acid solutions from 10 per cent to 30 per cent strength. In order to stop the etching the acid must be thoroughly washed with clean water from the surface of the concrete.

When mortar colors are used, they should not exceed 10 per cent by weight of the cement in the mixture. Coloring pigments should be very thoroughly mixed with the cement before either water or aggregate is added.

It is recommended that color effects be obtained as far as is possible by the use of exposed aggregates, because of the care required to secure rich and uniform shades by colored pigments. In general, more uniform results may be expected in the lighter colors. An effective use of colored pigments is found in producing mortar to blend with exposed aggregates.

Variations in the color of the materials, including the pigments themselves, are such as to make color formulas only approximate. Best results are obtained by experiment or trial. After selecting the primary color desired, the exact color may be determined by preparing a number of small mortar panels which should be made of the same materials and proportions as are intended to be used in the actual work. Panels will have a darker shade when wet than when dry.

The best method in products manufacture is to mix cement and pigment in a ball mixer. Pigment colored surfaces are to be protected from direct sunlight and kept moist for several days. The intensities of shades produced by mineral pigments will be increased by more thorough mixing of the mortar or concrete. Therefore care should be taken to secure a uniform time of mixing.

Manufacture of Light Concrete Building Tile.

Submitted by Committee P-6.

Practical application of the principles of products plant operation discussed in the 1924 report of Committee P-6 was made in a series of tests on 5 x 8 x 12-in. concrete building tile conducted at the plant of the Humboldt Gravel & Tile Co., Humboldt, Iowa, and the Structural Materials Research Laboratory, Lewis Institute, Chicago. The tests were made to determine

(1) Economical mixes for commercial use,
(2) Effect of using concrete at various intervals after mixing,
(3) Effect of mixing concrete part time dry and part time wet as compared with mixing continuously wet.

Outline of Tests.

Concrete to be mixed 6 minutes. Tile to be made from 1: 4 to 1: 9 mixes.

Materials to be proportioned by weight. Specimens to be steam cured.

Best quantity of water for each mix to be determined by trying several proportions of volume of water to volume of cement (water-cement ratio) for 1: 5 and 1: 8 mixes, and then calculating for other mixes.

Seven specimens to be made for each reference number, 5 specimens to be broken in compression and 2 specimens to be tested for absorption. Parallel tests to be made on 2 x 4-in. cylinders.

Materials.

Aggregate—Four commercial sizes known as No. 2, No. 5, Torpedo, and asphalt sand from gravel plant of Humboldt Gravel & Tile Co.

Cement—Universal Portland Cement in bulk.

Water—Well water as used in plant.

Grading of Aggregates.

A grading of aggregate (Fig. 7) which gave excellent results in the Cincinnati tests described in the 1924 report of Committee P-6 was used as a guide in determining the proportioning of the aggregates. In order to obtain a grading approximating that shown as "A", Fig. 7, (F.M. = 3.7), 4 available aggregates were used, in the following proportions, giving a fineness modulus of 3.7. Shown as "B", Fig. 7.

Per Cent	
60	No. 2 sand
20	Torpedo
10	No. 5 sand
10	Asphalt sand

Table 2 shows sieve analyses of all aggregates, some as actually made and others as calculated.

TABLE 1.—OUTLINE OF TESTS.

Reference No.	Mix by Dry Weight	Time of Mixing, minutes		Time Elapsed between Mixing of Concrete and Making of Tile, hours	Ratio of Volume of Water to Volume of Cement	Total Number of Specimens
		Dry	Wet			
1, 2, 3	1:5	3	3	0	1.0, 0.95, 0.90	21
4, 5, 6, 7, 8	1:5	3	3	0, ¼, ½, 1, 2	0.95	35
9, 10, 11	1:8	3	3	0	1.45, 1.35, 1.25	21
12, 24	1:4	3, 0	3, 6	0	0.805	14
13, 14, 15, 27	1:5	3, 3, 3, 0	3, 3, 3, 6	0, 1, 2, 0	0.95	28
16, 25	1:6	3, 0	3, 6	0	1.10	14
17, 18, 19	1:7	3	3	0, 1, 2	1.24	21
20, 26	1:8	3, 0	3, 6	0	1.39	14
21, 22, 23	1:9	3	3	0, 1, 1½	1.61	21
Grand Total						189

TABLE 2.—SIEVE ANALYSES OF AGGREGATES, PER CENT COARSER THAN A GIVEN SIEVE.

Sieve Size or Number	No. 2	Torpedo	No. 5	Asphalt Sand	"A" Figure 9 Desired	"B" Figure 9 Calculated	Grading Actually Used	
							December 1	December 2
100	99.0	100.0	100.0	83.0	96.0	98.0	97.0	97.0
48	95.0	99.0	100.0	48.0	86.0	92.0	90.0	92.0
28	80.0	98.0	100.0	16.0	72.0	79.0	77.0	77.0
14	31.0	97.0	100.0	1.0	56.0	48.0	51.0	46.0
8	16.0	91.0	99.0	0.0	39.0	38.0	41.0	37.0
4	3.0	21.0	91.0	0.0	20.0	15.0	22.0	14.0
⅜ in.	0.0	0.0	5.0	0.0	0.0	1.0	1.0	1.0
F. M.	3.24	5.06	5.95	1.48	3.69	3.71	3.79	3.64

The moisture content for each aggregate was

	Per Cent
No. 2	5.9 by weight of dry aggregate
Torpedo	5.5 " " " " "
No. 5	3.1 " " " " "
Asphalt sand	7.0 " " " " "

The calculated moisture content of the combined aggregates as used was 5.65 per cent by weight of dry aggregate.

A cubic foot of dry rodded mixed aggregate weighed 112 lb. A cubic foot of wet mixed aggregate measured loose weighed 93½ lb. As there was 5.65 per cent moisture in the aggregate, there was $\frac{93.5}{105.6} = 88.5$ lb. of dry aggregate in each cubic foot of wet mixed aggregate measured loose. To obtain 100 lb. of dry aggregate, 105.65 lb. of wet aggregate as used was required.

Following is the calculation to show how to determine proportions by dry rodded volume from the proportions by dry weight as used in these tests. For each batch 528 lb. of wet aggregate, equivalent to 500 lb. of dry aggregate, was used. For a 1: 5 mix by dry weight, 100 lb. of cement to 500 lb. of dry aggregate was used. Cement is figured at 94 lb. per cubic foot.

$$\frac{500}{112} = 4.464 \text{ cu. ft. of dry rodded aggregate.}$$

$$\frac{100}{94} = 1.064 \text{ cu. ft. of cement.}$$

$$1.064 : 4.464 = 1 : 4.2.$$

Therefore 1: 5 by dry weight was equivalent to 1: 4.2 by dry rodded volume.

$$\frac{4.2}{5} = 0.84.$$

Proportion by dry rodded volume, therefore, could be obtained by multiplying proportion by dry weight by 0.84. A proportion of 1: 9 by dry weight could equal 1: 9 x 0.84 = 1: 7.56 by dry rodded volume.

Proportions by volume measured wet and loose could be calculated as follows:

$$\frac{528}{93.5} = 5.65 \text{ cu. ft. wet aggregate measured loose.}$$

$$\frac{100}{94} = 1.064 \text{ cu. ft. of cement.}$$

$$1.064 : 5.65 = 1 : 5.31.$$

Therefore, 1: 5 by dry weight was equivalent to 1: 5.3 by volume measured wet and loose.

$$\frac{5.31}{5} = 1.062.$$

Therefore proportion by volume measured wet and loose could be obtained by multiplying proportion by dry weight by 1.062.

Mixing.

The concrete was mixed in an Ideal bottom dump, batch, paddle mixer, having a capacity of 14 cu. ft. loose material. Because of the size of the mixer, it was convenient to use 528 lb. of wet aggregate (500 lb. dry aggregate) in each batch. The quantities of cement and water were changed to give the proportions and workability desired. Each aggregate as well as the cement and water was weighed separately on a platform scale for each batch. The aggregates were put in the mixer first, then the cement and finally the water. Most of the tile were made from concrete mixed for 6 minutes after the water was added. The remainder of the tile were made from concrete mixed three minutes dry before the water was added and then three minutes after the water was added.

Test Pieces and Making of Tile.

Most of the concrete was used immediately after mixing but some of the concrete was set aside and made into tile one-half, one and two hours after mixing. In order to prevent loss of water, the concrete thus set aside was placed on a damp concrete floor and covered with damp burlap until used. The temperature of the work room was approximately 65 deg. F. The tile were made on an Ideal Roll-Over machine and were commercial 5 x 8 x 12-in. tile with cores vertical. The proportion of net area in bearing to gross area is 42 per cent. The regular plant force was used. Each tile was tamped with eight blows, the usual commercial practice with this machine. The size of the mold box allows two tile to be made simultaneously. The tile were put on racks as soon as made and taken into the steam room with a lift truck.

During the making of the tile it was noticeable that in the lean mixes the concrete under one tamp foot was cracking frequently. The bottom of this tamp foot did not have a uniform contact with the concrete. An analysis of the low breaks seems to show that this tamp foot was responsible for a number of breaks that are out of line with other breaks in the same set of tile. The average strength results can, therefore, be considered conservative and possible in regular plant operation.

Curing.

As the tile were made Dec. 1 and 2, they were cured for 21 days in a steam room at an average temperature of 70 deg. F., in order to ap-

proximate a favorable and uniform curing condition. This method of curing would tend to eliminate variations in the strength of tile left to cure out of doors and yet would not develop strengths that might be considered out of line with commercial practice. Temperature readings taken every 6 hours while the tile were in the curing room showed a range of temperature from 50 to 80 deg. F., averaging 70 deg. F. At the end of 21 days the moist tile were crated in straw and shipped to the Structural Materials Research Laboratory, Lewis Institute, Chicago, for test. On account of the zero weather which occurred while the tile were in transit,

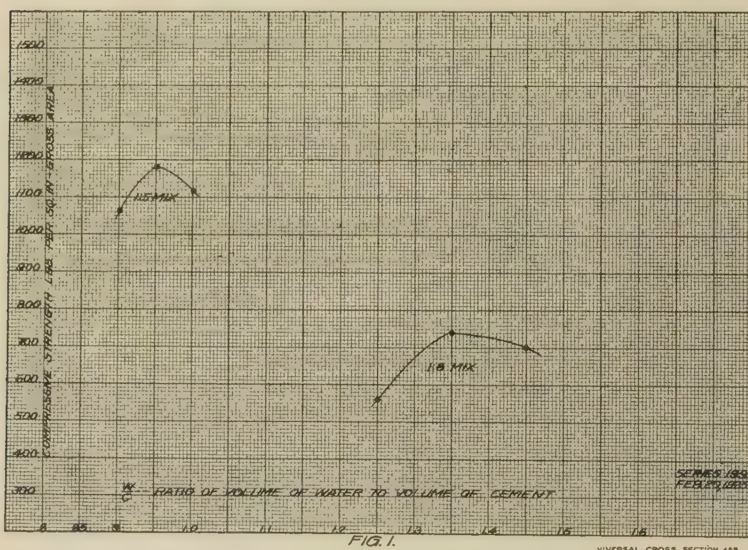

FIG. 1.—VALUES USED IN DETERMINING BEST WATER CEMENT-RATIO FOR BUILDING TILE.

they arrived at the laboratory in a frozen condition. The tile were, therefore, allowed to thaw and dry in the warm air of the laboratory at approximately 70 deg. F. for 4 days before being tested at the age of 32 days. The original intention was to break these tile at 28 days but the condition of the tile upon arrival at the laboratory necessitated holding them for the extra 4 days.

Testing.

The tile were tested in accordance with the 1924 Tentative Standard Specifications of the American Concrete Institute for concrete building tile. In general, 5 specimens in each group were tested for compressive strength, and 2 additional tile for absorption. A 200,000-lb. Universal testing machine was used for the compressive strength tests. Compressive tests were

also made on the 2 tile used in the absorption test in order to obtain information as to the effect of making an absorption test on a tile which is later to be tested in compression. The results of these compressive tests are given in Table 3, but are not included in summary and have not been used in making any of the diagrams. The values as tabulated and plotted are generally the average of 5 tile for the compressive strength and the average of 2 tile for absorption.

Two by Four-inch Cylinder Tests.

In order to compare the strength of tile with the strength of 2 x 4-in. cylinders, a sufficient quantity of the aggregates was sent to the Buffington, Indiana, laboratory of the Universal Portland Cement Co. to make parallel tests on the effect of varying water-cement ratios, and the effect of varying quantities of cement to aggregate. The same water-cement ratios and proportions of cement to aggregate as used in the tile series were used in the cylinder series. The cylinders were made in accordance with the specifications of the American Society for Testing Materials. They were kept in the moist closet for one day and then under water for 27 days before being tested at the age of 28 days. Each value as used is the average of 5 cylinders. See Fig. 6 and summary.

Discussion of Tests.

The proper quantity of water to use for each mix was determined by first deciding on the right quantity for a 1: 5 mix and also for a 1: 8 mix. From the Cincinnati tests on light concrete building tile, a definite relation had been determined between mix and the proportion of volume of water to volume of cement as expressed by the water-cement ratio for a specific aggregate. It was found that the maximum amount of water should be used which would allow the product to be stripped from the machine without bulging or slumping. With this relation as a basis, the best workabilities were easily secured by using quantities of water within which it was felt the best results both for strength and workability would be found. (See Fig. 1.) For the 1: 5 mix the quantities of water used were in proportions of 1.0, 0.95 and 0.90 to the volume of cement. The best workability was obtained with a proportion of 0.95. This quantity of water gave concrete, which when worked in a ball for half a minute as suggested by the 1924 report, showed a good film of moisture on the surface. For the 1: 8 mix, the quantity of water giving the best workability was in the proportion of 1.39, to volume of cement. This checked with the Cincinnati tests. A straight line relation (See Fig. 5) passing through 0.95 for a 1: 5 mix and 1.39 for a 1: 8 mix gives 0.805 for a 1: 4 mix, 1.10 for a 1: 6 mix, 1.24 for a 1: 7 mix, 1.53 for a 1: 9 mix. In making the complete series of tile beginning with the 1: 4 and using the quantities of water as determined by the proportion of water to cement for each mix it was found that the quantity of water for the 1: 8 mix gave a tile that was a trifle dry. This indicated that the 1: 9 mix on the same basis would

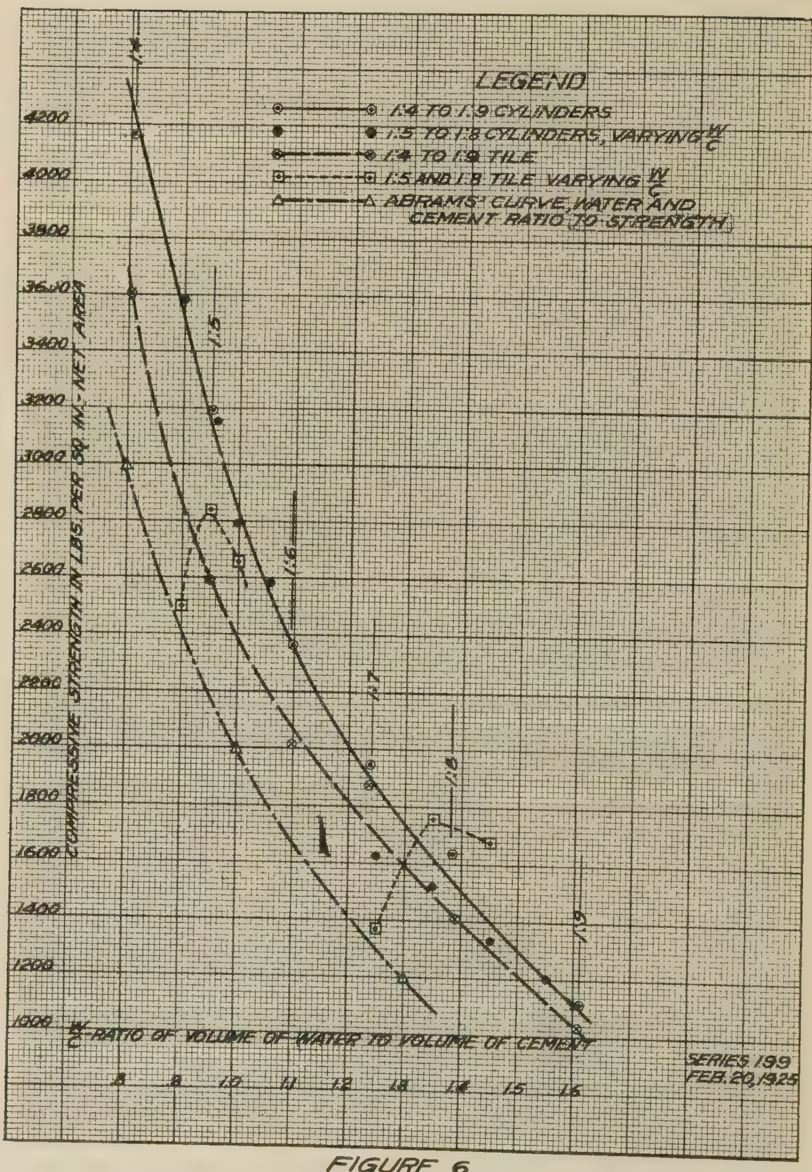

FIG. 6.—COMPARISONS OF STRENGTHS OF TILE AND 2 BY 4 INCH CYLINDERS HAVING SAME AGGREGATE, WATER AND CEMENT CONTENT.

be too dry and it was, therefore, decided to add 5 per cent to the calculated quantity of water giving a quantity of water equivalent to a proportion of 1.61 for making the 1: 9 tile. This quantity of water gave concrete that worked satisfactorily in the machine.

It is interesting to note that the quantity of water which gave best workability also gave the best strength. For the 1: 5 mix, the right quantity of water, equivalent to a water-cement ratio of 0.95, gave a strength of 1185 lb. per square inch of gross area. The wetter 1 water-cement ratio gave 1,120 lb. per square inch of gross area and the drier 0.90 water-

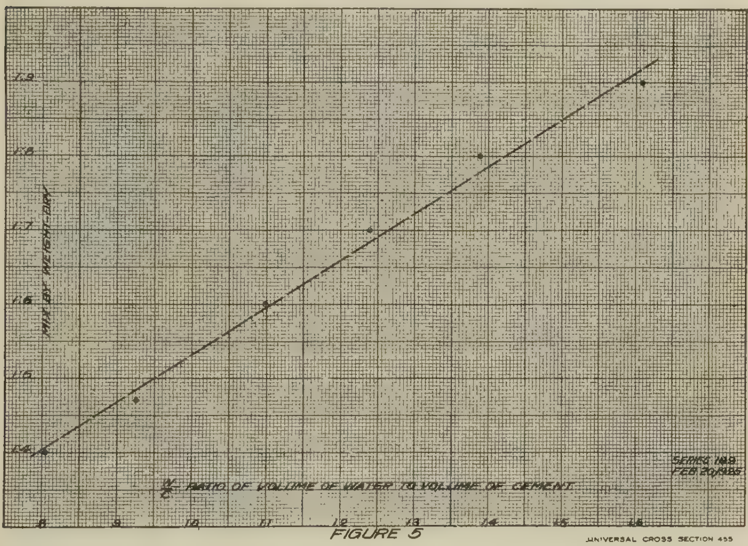

FIG. 5.—RELATION BETWEEN WATER-CEMENT RATIO AND MIX FOR FINENESS MODULUS = 3.7.

cement ratio gave 1,060 lb. per square inch of gross area. In the parallel 2 x 4-in. cylinder tests, however, the 0.90 water-cement ratio gave a strength 13 per cent above the strength of the 0.95 water-cement ratio. This is no doubt due to the fact that more and better tamping is possible on the cylinder than on the commercial tile, and a concrete that is too dry and is not workable on a tile machine can be made workable by tamping in a cylinder. Similar results as to workability were noticeable in the 1: 8 series to determine best water-cement ratio.

Effect of an Interval of Time Between Mixing of Concrete and Making of Tile.

In this series, concrete for each mix was used from a single batch, being divided equally into sufficient amounts so as to make at least six tile

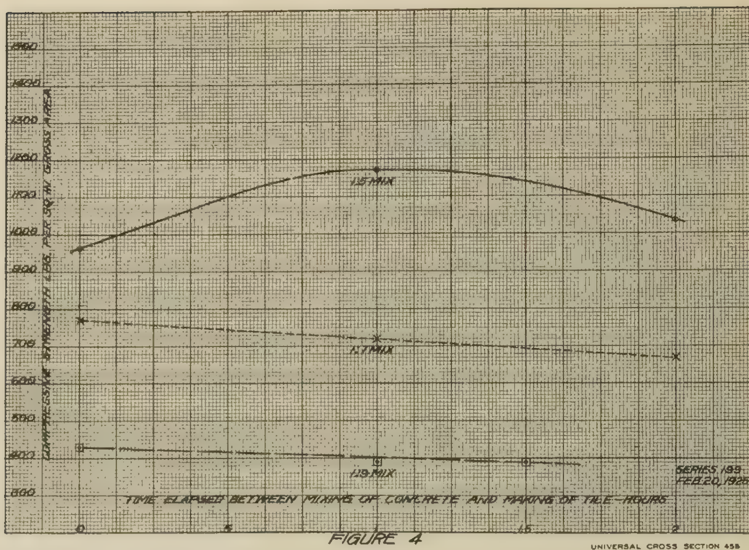

FIG. 4.—EFFECT OF AN INTERVAL OF TIME BETWEEN MIXING OF CONCRETE AND MAKING OF TILE.

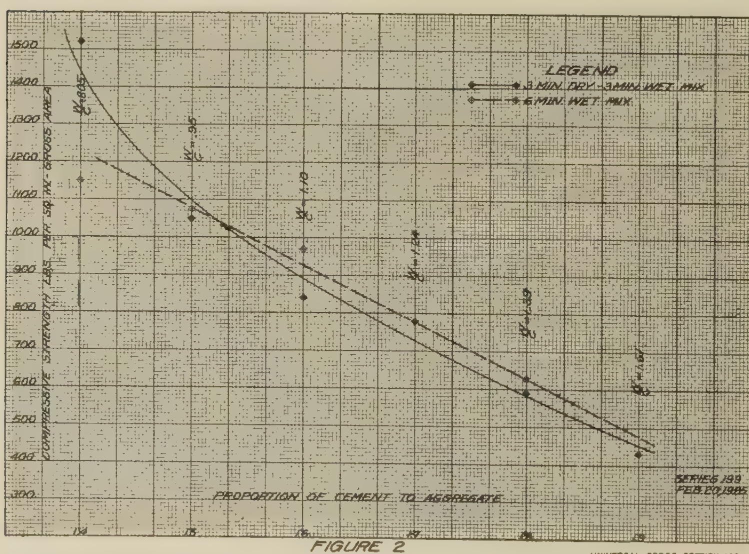

FIG. 2.—EFFECT OF DIFFERENT METHODS OF MIXING.

for each condition. Some of the tile were made immediately, others in one-half hour, one hour and two hours. The time elapsed between mixing of concrete and making of tile seemed to have no noticeable effect on the workability of the concrete in the machine. The strengths in general decreased slightly the longer the elapsed time. For the 1: 5 mix there was an apparent increase of 6 per cent on tile made at end of two hours over the tile made immediately. For the 1: 7 mix, there was a decrease of 14 per cent for the tile made at the end of two hours compared to tile made immediately, and for the 1: 9 mix a decrease of 10 per cent. There is

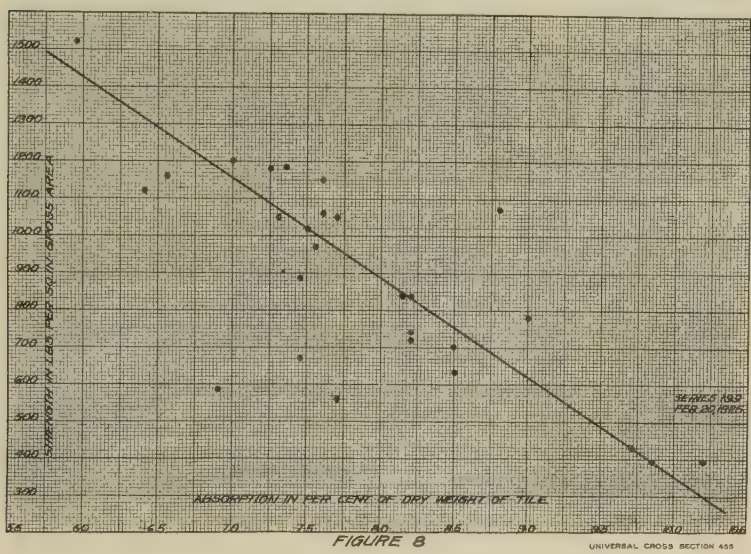

FIG. 8.—RELATION BETWEEN STRENGTH AND ABSORPTION.

apparently no advantage or disadvantage in allowing concrete to remain unused for one or two hours after mixing. (See Fig. 4.)

Effect of Method of Mixing.

All but four sets in the series were made from concrete mixed six minutes wet. These four sets were made from 1: 4, 1: 5, 1: 6, 1: 8 mixes and were mixed three minutes dry and three minutes wet. The results using the two methods of mixing were generally close. However the 1: 4 concrete mixed six minutes wet had a strength of 1,520 lb. per square inch of gross area, and the 1: 4 concrete mixed three minutes dry and three minutes wet had a strength of only 1,150 lb. per square inch of gross area. Not enough tests were made to prove that this relation for the 1: 4 mix is in general true. The results would seem to indicate that by the end of

six minutes the concrete has been so thoroughly mixed that the effect of dry and wet mixing during the first three minutes is not noticeable. (See Fig. 2.)

Strength and Absorption Tests.

Though the absorption results are not as uniform as the strength results, a straight-line relation between strength and absorption was found when all of the average values in the entire 27 tests were plotted. (See Fig. 8.) An absorption of 9.5 per cent corresponded with a strength of

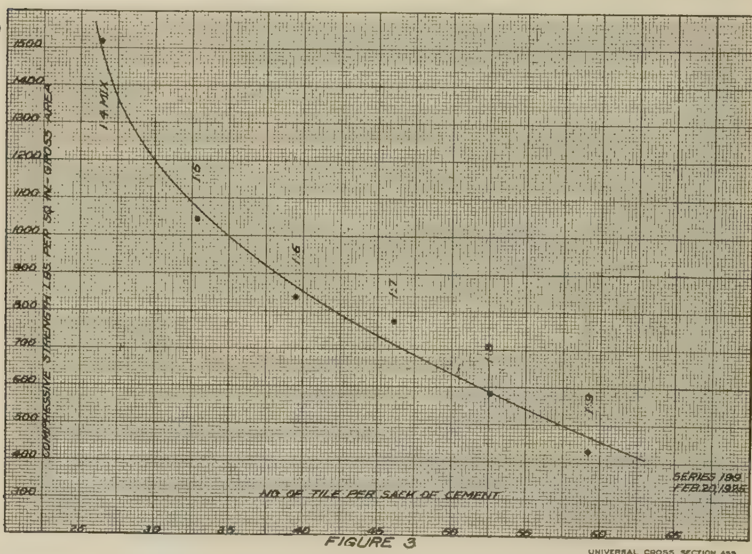

FIG. 3.—RELATION BETWEEN NUMBER OF TILE PER SACK OF CEMENT AND STRENGTH OF TILE.

500 lb. per square inch of gross area, an absorption of 7.5 per cent with a strength of 1,000 lb. per square inch of gross area, and an absorption of 5.75 per cent with a strength of 1,500 lb. per square inch of gross area.

The relation between strength and water-cement ratio for tile (Fig. 6) is similar to the relation between strength and water-cement ratio for concrete in general, developed by Professor Abrams, Bulletin 1, Structural Materials Research Laboratory, p. 3. The values in the tile series are approximately 400 lb. per square inch net area higher than the values in Bulletin 1 for similar water-cement ratios. This is probably due to the fact that the concrete in the tile is tamped while the concrete in the Structural Materials Research Laboratory tests is rodded. At the Buffington, Ind., laboratory, where the cylinders for these tests were made, the concrete was tamped.

Number of Tile Per Sack of Cement.

Though no effort was made to calculate accurately the number of tile made from each batch of concrete, it was found that on the average 35 tile were easily obtained per batch. On this assumption, calculation was made as to the number obtained per sack of cement. Fig. 3 shows the relation between the number of tile per sack of cement and strength of tile. Assuming a desired strength of 780 lb. per square inch of gross area,

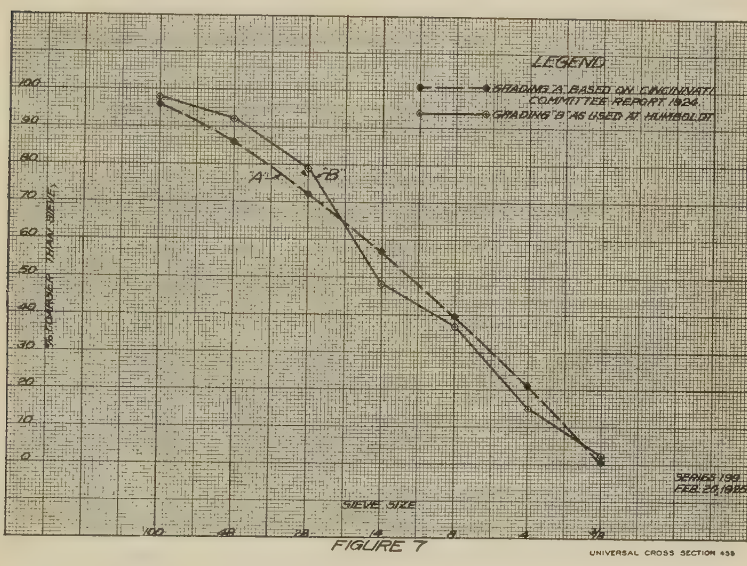

FIG. 7.—CURVE OF GRADING AS USED.

46 tile can be obtained per sack of cement. For a strength of 1,500 lb. per square inch of gross area, 25 tile can be obtained per sack of cement.

Comparison of Strengths of Tile Broken After Being Tested for Absorption and Strengths of Tile Broken Without Absorption Test.

The two specimens from each group that were tested for absorption were allowed to dry for five days after the absorption test and were then tested in compression at an age of 37 days. Leaving out of consideration five of the 27 groups that were quite evidently out of line, the strength of the five tile from each group tested at 32 days without absorption averaged 2.8 per cent more than the strength of the 2 tile from each group tested at 37 days after being put through the absorption test five days previous.

Table 3 is a comparison between the strength of the tile tested at 32 days without being put through the absorption test and the tile tested at 37 days after drying five days after being put through absorption test.

TABLE No. 3.
COMPRESSIVE STRENGTH.
Lbs. per sq. in. Gross Area.

Reference Number	Tested without Absorption at 32 days	Tested after Absorption Test at 37 days†
1	1,120	1,265
2	1,185	1,195
3	1,060	1,085
* 4	885	1,145
5	1,160	1,140
* 6	840	660
7	1,200	1,080
8	1,050	1,140
9	700	570
10	740	620
*11	560	1,230
12	1,520	1,200
13	1,050	1,160
14	1,180	1,120
*15	1,020	725
16	840	900
*17	780	590
18	720	640
19	670	790
20	588	480
21	430	360
22	390	390
23	390	420
24	1,150	1,080
25	970	940
26	630	690
27	1,070	1,000

Cylinder Tests.

The predetermined water-cement ratios worked very well in making the cylinders. Good water marks appeared on all of the cylinders. Because of more tamping, dryer consistencies can be made workable in the cylinders than in the tile. Certain water-cement ratios that gave work-

*Inconsistent—not used in arriving at average results.
†Allowed to dry 5 days.

abilities too dry for getting maximum strength in the tile, gave high strengths in cylinders because the concrete could be tamped more vigorously in making the cylinders than in making the tile. The strength of the 5 x 8 x 12-in. tile averaged 10 per cent less than the strength of the 2 x 4-in. cylinders for the lean mixes and 14 per cent less for the rich mixes.

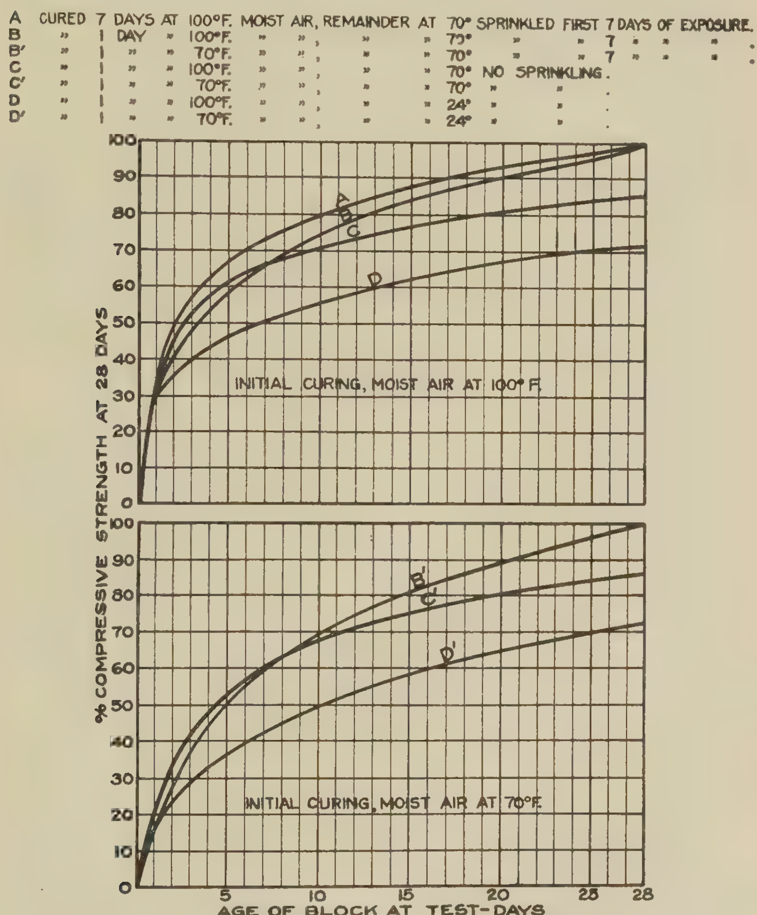

FIG. 7.—EFFECT OF INITIAL CURING AND SUBSEQUENT STORAGE ON 28-DAY COMPRESSIVE STRENGTH OF CONCRETE BLOCK.

A—Cured 7 days at 100° F. moist air, remainder at 70° F. sprinkled first 7 days of exposure; B—Cured 1 day at 100° F. moist air, remainder at 70° F. sprinkled first 7 days of exposure; B′—Cured 1 day at 70° F. moist air, remainder at 70° F. sprinkled first 7 days of exposure; C—Cured 1 day at 100° F. moist air, remainder at 70° F. no sprinkling; C′—Cured 1 day at 70° F. moist air, remainder at 70° F. no sprinkling; D—Cured 1 day at 100° F. moist air, remainder at 24° F. no sprinkling; D′—Cured 1 day at 70° F. moist air, remainder at 24° F. no sprinkling.

SUMMARY.

TABULATION OF RESULTS OF TESTS OF CONCRETE BUILDING TILE.

Tile made in plant of Humboldt Gravel & Tile Co., Humboldt, Iowa.
Ideal roll-over machine, Power tamper, Ideal mixer.
Curing 21 days in steam at 70° F., 7 days in transit at 10° F., 4 days in dry air of Laboratory at 70° F.
Age of tile at time of test 32 days.
Aggregate, a combination of 4 sizes, giving fineness modulus of 3.7.
Strength results in general are average of 5 specimens.
Absorption results in general are average of 2 specimens.
All tile tests made at Structural Materials Research Laboratory, Lewis Institute, Chicago.
All cylinder tests made at Buffington, Ind., Laboratory, Universal Portland Cement Co.

Reference No.	Mix by Dry Weight	Ratio of Volume of Water to Volume of Cement	Time of Mixing, minutes		Time Elapsed Between Mixing of Concrete and Making of Tile, hours	Average Weight of Tile, lbs.		Absorption, per cent	Compressive Strength of Tile, lb. per sq. in.		Compressive Strength of Cylinders, lb. per sq. in.
			Dry	Wet		Dried to Constant Weight	After Immersion for 24 Hours		Gross Area	Net Area	
1	1:5	1.0	3	3	0	16.14	17.18	6.4	1,120	2,650	2,783
2	1:5	0.95	3	3	0	16.16	17.34	7.35	1,185	2,850	3,183
3	1:5	0.90	3	3	0	15.90	17.11	7.6	1,060	2,510	3,587
4	1:5	0.95	3	3	0	15.75	16.92	7.45	885	2,035	....
5	1:5	0.95	3	3	¼	15.88	16.88	6.55	1,160	2,750	....
6	1:5	0.95	3	3	½	14.96	16.18	8.15	840	1,990	....
7	1:5	0.95	3	3	1	16.08	17.20	7.0	1,200	2,850	....
8	1:5	0.95	3	3	2	15.98	17.21	7.7	1,050	2,490	....
9	1:8	1.45	3	3	0	15.30	16.60	8.5	700	1,670	1,325
10	1:8	1.35	3	3	0	15.36	16.61	8.2	740	1,750	1,503
11	1:8	1.25	3	3	0	15.99	17.22	7.7	560	1,350	1,616
12	1:4	0.805	3	3	0	16.50	17.48	5.9	1,520	3,600	4,165
13	1:5	0.95	3	3	0	15.86	17.02	7.3	1,050	2,480	3,183
14	1:5	0.95	3	3	1	16.05	17.20	7.25	1,180	2,810	....
15	1:5	0.95	3	3	2	15.23	16.36	7.5	1,020	2,420	....
16	1:6	1:10	3	3	0	15.67	16.96	8.2	840	2,010	2,355
17	1:7	1.24	3	3	0	14.72	16.04	9.0	780	1,860	1,939
18	1:7	1.24	3	3	1	15.16	16.40	8.2	720	1,700	....
19	1:7	1.24	3	3	2	15.41	16.50	7.45	670	1,680	....
20	1:8	1.39	3	3	0	15.41	16.48	6.9	588	1,390	1,620
21	1:9	1.61	3	3	0	14.79	16.24	9.7	430	1,020	1,098
22	1:9	1.61	3	3	1	14.66	16.16	10.2	390	960	....
23	1:9	1.61	3	3	1½	14.56	16.00	9.85	390	930	....
24	1:4	0.805	0	6	0	16.35	17.60	7.65	1,150	2,720	....
25	1:6	1.10	0	6	0	15.62	16.80	7.55	970	2,320	....
26	1:8	1.39	0	6	0	15.25	16.55	8.5	630	1,500	....
27	1.5	0.95	0	6	0	16.05	17.47	8.8	1,070	2,550	....

Conclusions.

(1) Concrete which gave best workability for tile gave the greatest strength. (Fig. 1.)

(2) With a total of six minutes mixing, there was no advantage or disadvantage in mixing the concrete dry for half of this time. (Fig. 2.)

(3) With accurate control of manufacture and curing, a mix of 1:7 by dry weight produced 5 x 8 x 12-in. tile having a strength of more than 700 lb. per square inch of gross area at 28 days, which is the requirement of the standard specifications of the American Concrete Institute for medium load bearing tile. This is equivalent to 46 tile per sack of cement. (Figs. 3 and 6.)

(4) There was no advantage or disadvantage in using concrete when a period of time had elapsed after mixing, provided the elapsed time did not exceed 2 hours and the concrete was covered with damp burlap to prevent evaporation of water from the concrete. (Fig. 4.)

(5) A straight-line relation was found between mixes and the proportion of volume of water to volume of cement (water-cement ratio) which gave the best workability. (Fig. 5.)

(6) Strength of 5 x 8 x 12-in. tile made in the factory was approximately 90 per cent of the strength of 2 x 4-in. cylinders made of like concrete in the laboratory. (Fig. 6.)

(7) The relation between proportion of volume of water to volume of cement (water-cement ratio) and strength, developed at the Structural Materials Research Laboratory, held true in general for the tile made in this test. (Fig. 6.)

(8) A calculated fineness modulus was easily adhered to by using several sizes of aggregate and by weighing the material. (Fig. 7.)

(9) There was a direct relation between the strength of tile and absorption for the same aggregate. (Fig. 8.)

Investigation of the Effect of Various Methods of Curing Concrete Building Block.

Submitted by Committee P-6.

Introduction.

This report covers compression tests on two hundred and ninety-seven 8 x 8 x 16-in. 3-oval-core hollow concrete building block made in a commercial plant and cured under different conditions. The tests were outlined and carried out by the Subcommittee on Curing of Committee P-6 of the American Concrete Institute for the purpose of studying the effect of temperature and moisture on the compressive strength of concrete block at ages up to 28 days. The block were manufactured and given their initial curing at the plant of Bert Carey and Co., Forest Park, Ill., and were then moved to the Structural Materials Research Laboratory, Lewis Institute, Chicago, for further curing and for testing. Acknowledgment is made to the Bert Carey Co. and to the Structural Materials Research Laboratory for the valuable and generous co-operation afforded the subcommittee in carrying out these tests.

This investigation is limited in scope and should be considered as preliminary to a more comprehensive series of tests on curing which the committee plans to carry out as soon as conditions permit.

The tests were designed to obtain information on the following:

Group 1: 28-day compressive strength of 1 : 7 and 1 : 4 concrete block cured 1 to 7 days in moist air at 100 deg. F., remainder in

- (a) air of laboratory,
- (b) air of laboratory sprinkled once daily for 7 days,
- (c) moist room in laboratory,
- (d) air outdoors exposed to weather.

Group 2: 1- 2- 3- and 7-day compressive strength of 1 : 7 and 1 : 4 concrete block cured in moist air of 100 deg. F.

Group 3: 28-day compressive strength of 1 : 7 and 1 : 4 concrete block cured for 2 days in either dry or semi-moist air at 100 deg. F., remainder as in Group 1.

Group 4: 28-day compressive strength of 1 : 10, 1 : 7 and 1 : 4 concrete block cured for 2 days in either dry or moist air at 70 deg. F., or in moist air at 130 deg. F., remainder as in Group 1.

Group 5: 1- 2- 3- 7- and 28-day compressive strength of 1 : 7 and 1 : 4 concrete block cured in dry air of 70 deg. F.

In general, three block were made for each condition of test.

Scope of Tests.

An outline of the tests covered by this investigation is given in Table 1 herewith:

TABLE 1.—OUTLINE OF TESTS.

Compression tests of 8 x 8 x 16-in. 3-core concrete building blocks.
Mix by volume (based on dry and rodded aggregate).
Cement: Portland.
Aggregate (F. M. 3.70) a mixture of 0 to No. 8 sand, 0 to No. 8 limestone screenings and No. 4 to ⅝-in. crushed limestone.
Consistency as wet as practicable.
Machine-mixed concrete.
Blocks cured as indicated.
In general, 3 specimens made for each condition from different batches.
Blocks tested at 28 days were air-dried in Laboratory for 3 days before test. Block tested at less than 28 days were in condition as taken from storage.

Group	Reference No.	Mix	In Curing Room: Temperature of Curing Room, deg. F.	In Curing Room: Condition of Air of Curing Room	In Curing Room: Time in Curing Room, days	In Curing Room: Curing Treatment upon Removal from Curing Room	Age at Test, days	Total Specimens
1	1–16 17–32	1:7 1:4	100	Moist....	1 2 3 7	(a) Air of Laboratory.......... (b) Air of Laboratory sprinkled once daily for 7 days..... (c) Moist room in Laboratory.. (d) Air outdoors exposed to cold weather............	28	96
2	33–36 37–40	1:7 1:4	100	Moist....	1 2 3 7	Tested at once............	1 2 3 7	24
3	41–44 49–52 45–48 53–56	1:7 1:4	... ...	Dry........ Semi-moist..	2	(a) Air of Laboratory.......... (b) Air of Laboratory sprinkled once daily for 7 days..... (c) Moist room in Laboratory.. (d) Air outdoors exposed to cold weather............	28	48
4	115–118 119–122 123–126 57–60 73–76 97–100 61–64 77–80 101–104	1:10 1:7 1:4	70 130*	Dry........ Moist.......	2	(a) Air of Laboratory.......... (b) Air of Laboratory sprinkled once daily for 7 days..... (c) Moist room in Laboratory.. (d) Air outdoors exposed to cold weather............	28	104
5	105–108 110–114	1:7 1:4	70	Dry........	2	In air of Laboratory.........	1† 2 3 7 28	25
Total.	....	......	....	....	......	....	......	297

* Block cured at 130° F. made for moist air curing only.
† Block removed from curing room after 1 day.

All block tested at 28 days were allowed to dry for at least three days in the laboratory before testing.

A total of 297 block were tested.

Materials and Methods.

Aggregates: Three different aggregates were used in making the block as follows:

Sand, 0 to No. 8.
Limestone screenings, 0 to No. 8.
Crushed limestone, No. 4 to 3/8.

The 0 to No. 8 sand and 0 to No. 8 screenings were taken from the aggregate bins of the block plant; the No. 4 to 3/8-in. crushed limestone was purchased in the local market for use in the tests in order to secure a mixed aggregate graded from 0 to 3/8 in.

The aggregates were combined in such proportions as to give a fineness modulus of the mixed aggregate of 3.8. The mixed aggregate used produced concrete of entirely satisfactory workability for the type of machine employed and the surfaces of the test block were smooth. The average sieve analyses and the miscellaneous tests on several samples of each aggregate, selected during the time the block were being made are given in Table 2.

TABLE 2.—SIEVE ANALYSES AND MISCELLANEOUS TESTS OF AGGREGATES.

Torpedo sand and limestone screenings from stock piles at plant of Bert Carey & Co., crushed limestone purchased in local market for use in curing tests.

SIEVE ANALYSIS.

Each sieve analysis is the average of 4 tests made on different days.

Number or Size of Sieve	Size of Square Opening, inches	Amount Coarser than Each Sieve, per cent by weight		
		Sand	Limestone Screenings	Crushed Limestone
100	0.0058	98	87	100
48	0.0117	88	79	100
28	0.0232	55	64	100
14	0.0469	26	44	100
8	0.093	12	18	98
4	0.185	4	5	96
3/8 in.	0.37	0	0	2
3/4 in.	0.75	..	..	0
Fineness Modulus*	....	2.83	2.97	5.96

* Sum of the percentages in the sieve analysis, divided by 100.

MISCELLANEOUS TESTS.

	Sand	Limestone Screenings	Crushed Stone	Mixed Aggregate
Unit Weight, lb. per cu. ft., dry and rodded	112	103	92	115.5
Organic Impurities, by colorimetric test	...	...	...	...
Silt, per cent by weight	0.3	13.7	...	...

Cement: Universal portland cement from the supply of the block plant was used.

Mixing Water: The water for mixing the concrete was well water from the Forest Park city water supply.

Concrete: The proportions of cement to mixed aggregate for most of the block were 1: 7 and 1: 4 by volume. A few block were made of 1: 10 mix. The proportions were based on dry and rodded aggregate mixed as used. For example, a batch of 1: 4 concrete consisted of 94 lb. of cement (1 cu. ft. or one sack) and the equivalent of 4 cu. ft. of dry and rodded mixed aggregate. (See Table 3.) The quantities were weighed on a platform scale. The capacity of the mixer would not permit the use of a one-sack batch for the 1: 7 and 1: 10 mixes and for these mixes half-sack batches were used; for the 1: 4 mix one-sack batches were used.

The mixing water for each batch was also weighed. Moisture determinations were made on representative samples of the aggregate on each day of manufacture of block. The water held by the aggregate was taken into consideration in calculating the quantities of aggregate and water for a batch. Table 3 gives the quantities of materials, proportion of water to volume of cement, and the number of block produced per sack of cement for the different mixes.

TABLE 3.—QUANTITIES OF MATERIALS PER BATCH USED IN MANUFACTURE OF TEST BLOCKS.

Water ratios and block made per sack of cement.

Item	Average Quantities for Different Mixes		
	1:10	1:7	1:4
Cement, lb.	94	94	94
0 to No. 8 sand, lb.	454	328	188
0 to No. 8 screenings, lb.	446	304	174
No. 4 to ⅜-in. crushed stone, lb.	257	179	101
Water { in aggregate, lb.	35	27	15
Water { added, lb.	53.0	46.6	33.2
Water { total, lb.	88.0	73.6	47.4
Water ratio	1.40	1.17	0.76
Blocks per sack of cement	94	18	11

Consistency: The consistency of concrete used was as wet as practicable. The characteristic web marks were visible on the surfaces of the block upon removal from the machine but no sagging of the block occurred.

Mixing: The concrete was mixed in a bottom-dump Blystone mixer of about 5 cu. ft. capacity. The average time of mixing for each batch was 4 minutes; 1½ minutes before water was added and 2½ minutes after water was added. When mixed, the concrete was discharged into a chute leading to the block machine.

Making of Block: The block were of the 3-core type and were 8 x 8 x 16-in. in size. They were made on a Universal automatic power tamping machine, run by an experienced operator regularly employed at the plant. The usual plant practice was followed. (The cores were stripped horizontally.)

Upon removal from the machine the block were placed on a truck and moved to the curing room.

The schedule of tests was so arranged that block for only one condition of curing could be made on a given date. The block were made on six different days over the period Dec. 1, 1924, to Jan. 5, 1925.

In general, 3 block were made for each condition of tests. Enough block of each mix were made on each day of manufacture to furnish all the specimens required for the different conditions of test. The 3 block of similar mix for a given condition of test were selected at random from the day's run. Low temperatures were encountered during the period of manufacture and curing of the block. The mean daily out-door temperature ranged from 44 deg. to — 13 deg. F.

Curing: The block were given their initial curing in a specially equipped curing room at the plant set aside for the tests. This room was about 4 ft. wide, 7 ft. high and 30 ft. long and was equipped with a pipe-coil radiator attached to one side wall. Steam for furnishing moisture was introduced into the curing room through a perforated pipe along the side wall. An inside curing room subjected to minimum exposure was used for the tests. The curing room was open at one end only. The opening was directly into the work room of the plant. The opening was provided with a heavy canvas curtain.

Temperatures in the curing room were maintained as nearly as practicable at 70, 100 or 130 deg. F. and the attempt was made to have the air of the curing room:

(1) Saturated with moisture (designated as moist in Tables and Diagrams).
(2) Partially moistened (semi-moist).
(3) With practically no moisture (dry).

Moisture regulation was secured by manipulation of a valve on the exhaust steam line leading to the perforated pipe in the curing room. Due to the small amount of water required to saturate the air in the curing room, moisture regulation was not very successful. Enough water to saturate the air was evidently evaporated from the block in the room without affecting the strength of the block. It is calculated that only 1 lb. of water at a temperature of 70 deg. F., 2.4 lb. at 100 deg. F. and 5.30 lb. at 130 deg. F. was necessary to saturate the air in the curing room.

Upon removal from the curing room the block were taken to the laboratory and either tested the same day they were received or given additional curing as follows:

(a) air of laboratory (mean temperature about 70 deg. F.),

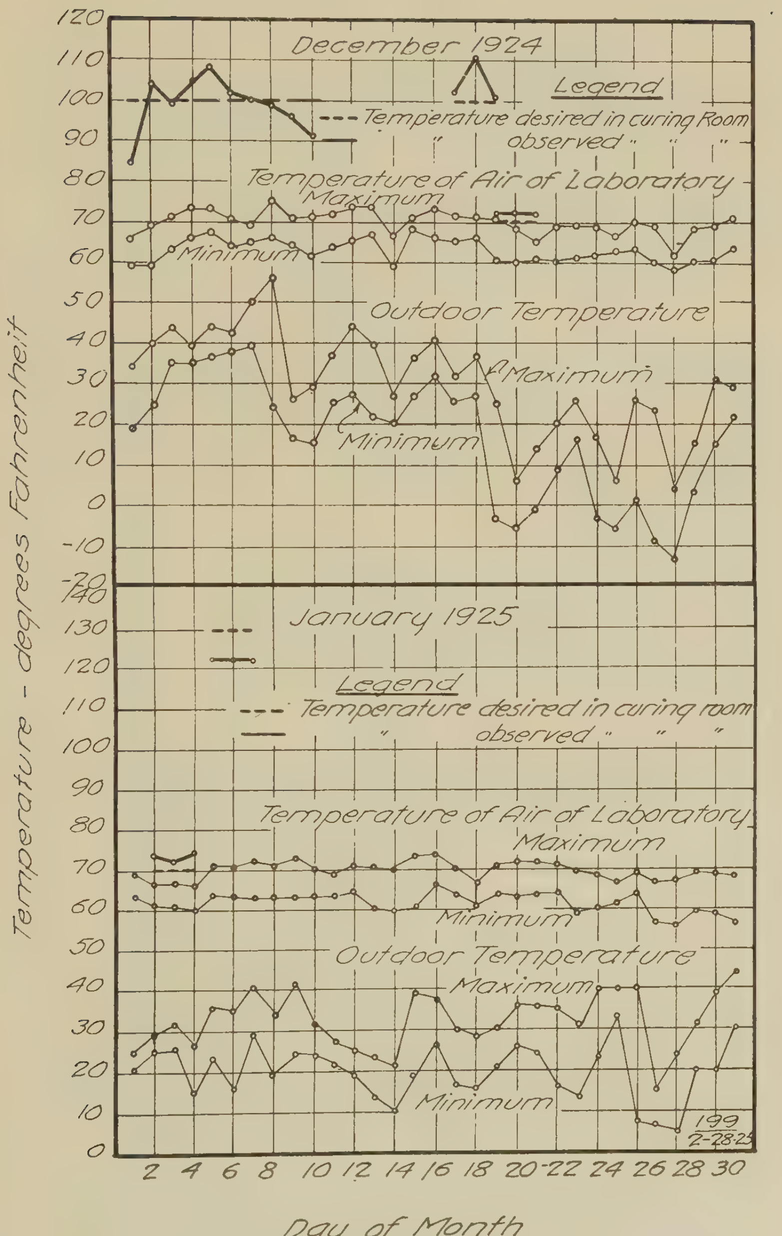

FIG. 1.—TEMPERATURE DURING CURING OF CONCRETE BLOCK.

Outdoor Temperatures are from the Monthly Meterological* Summary for the Chicago Station of U. S. Weather Bureau. Temperatures of air of Laboratory are from records of Laboratory.

(b) air of laboratory sprinkled once daily for 7 days (mean temperature of 70 deg. F.),
(c) moist room (mean temperature about 55 deg. F.),
(d) outdoors exposed to weather (mean temperature about 24 deg. F.).

Temperatures are shown in Fig. 1. The moist room temperatures at the laboratory averaged about 55 deg. F.

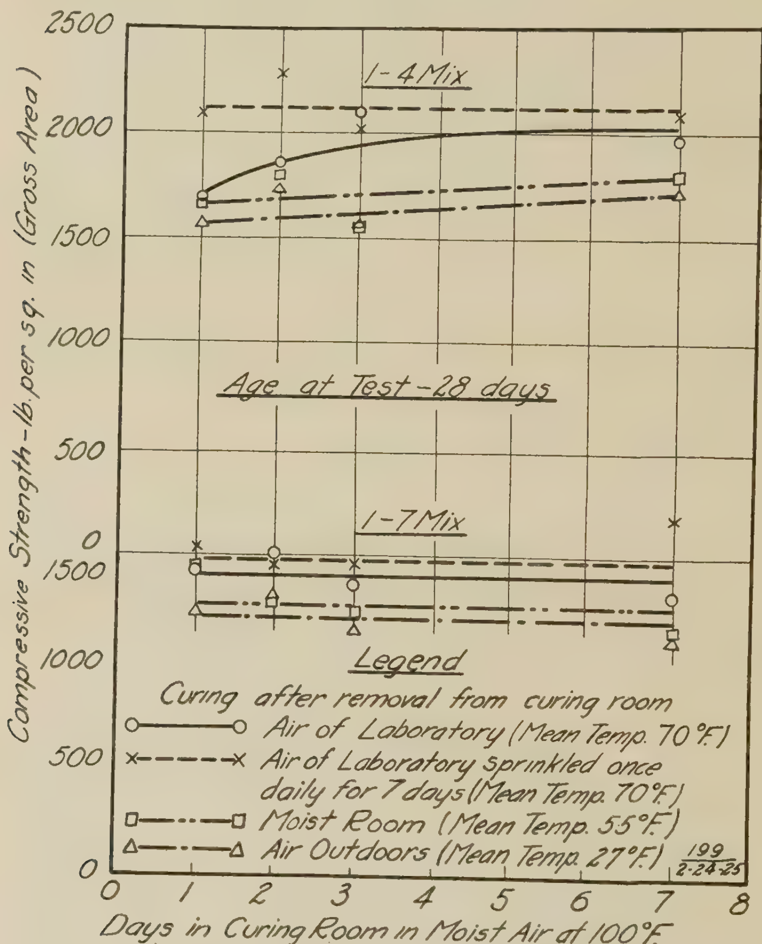

FIG. 2.—EFFECT OF CURING AT 100 DEG. F. IN MOIST AIR ON THE 28-DAY COMPRESSIVE STRENGTH OF CONCRETE BLOCK.

Compressive tests of 8 x 8 x 16-in. block. Mix by volume. Block cured in room in moist air at 100 deg. F. for 1, 2, 3, and 7 days, remainder as indicated. Each value is the average of 3 tests. Data from Table 4.

Tests and Methods of Testing.

Compression tests were made on the block as laid in the wall at ages of 1, 2, 3, 7 and 28 days. Block cured in moist room and outdoors and tested at 28 days were air-dried in the laboratory for 3 days before testing. The loaded surfaces of the block were capped with a thin layer of a mixture of equal parts of gypsum and portland cement. In testing a spherical bearing block was placed on a heavy steel distributing plate on top of the specimen in order to insure an even distribution of the load.

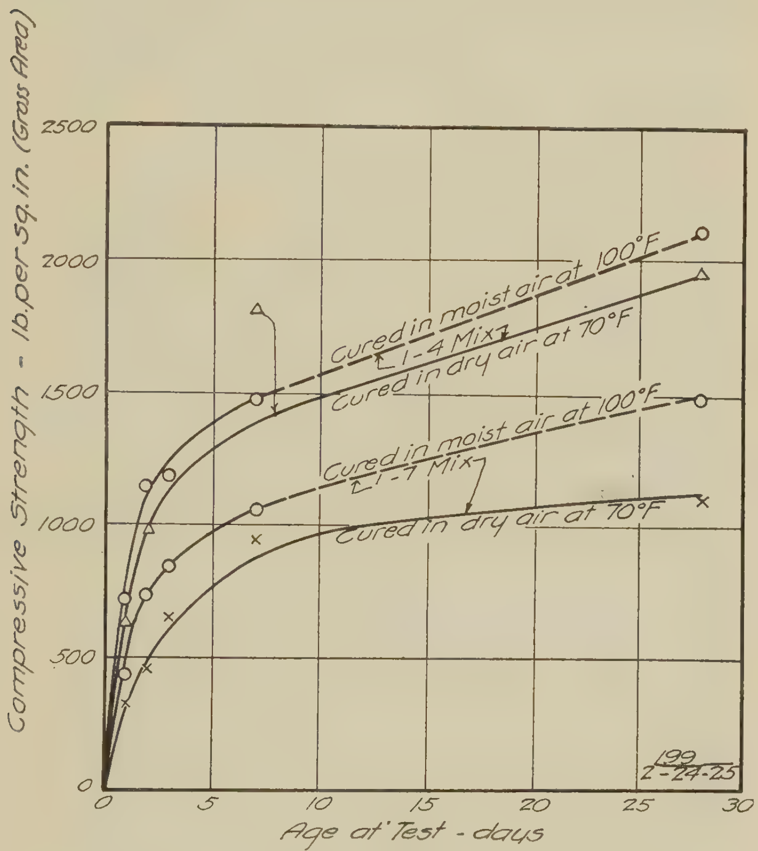

FIG. 3.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE BLOCK AS INFLUENCED BY CURING.

Compressive tests of 8 x 8 x 16-in. block. Mix by volume. Each value is the average of 3 tests. Data from Table 4.

Several of the block of 1: 4 mix which could not be loaded to failure in a 300,000-lb. testing machine were cut in half with a chisel and the half block tested.

Absorption tests were made on several block in accordance with the 1924 Tentative Standard Specifications for Concrete Building Block and Concrete Building Tile of the American Concrete Institute. Block of 1: 7 mix showed an average absorption of 6.7 per cent and block of 1: 4 mix an average absorption of 5.8 per cent.

Data and Discussion.

The compressive strengths of the block for the different conditions of test are given in Table 4. Both gross-area and minimum-area strengths are reported. The principal data are plotted in Fig. 2 and 7.

Fig. 2 shows the 28-day gross area strengths of 1: 4 and 1: 7 block cured in moist air of 100 deg. F. at the plant for 1 to 7 days followed by storage in

(a) air of laboratory,
(b) air of laboratory sprinkled once daily for 7 days,
(c) moist room in laboratory,
(d) outdoors exposed to cold weather.

In these tests there was no apparent increase in the *28-day strength* of 1: 4 block cured 7 days in moist air of 100 deg. F. over that of similar block cured only 1 or 2 days in moist air of 100 deg. F. previous to storage under any of the four conditions. (See Fig. 2.) Taking the 1: 4 and 1: 7 block tests together there appeared to be no advantage at 28 days in curing more than one day in moist air at 100 deg. F. before exposing the block to the air of the laboratory or in the moist room or out of doors in cold weather.

For both the 1: 4 and 1: 7 block the highest strengths were secured from the block which were sprinkled for 7 days after removal from the curing room. The block cured outdoors (mean daily temperature 27 deg. F.) gave about 20 per cent the lower strength.

The relative strengths of block cured in moist air at 100 deg. F. until tested at 1, 2, 3 or 7 days as compared to block tested at 28 days were:

Percentage of 28-Day Strength.

Mix	1 Day	2 Days	3 Days	7 Days	28 Days
1: 7	28	48	56	69	100
1: 4	34	54	56	70	100
Average ..	31	51	56	70	100

The curves in Fig. 3 show the effect on the compressive strength of 1: 4 and 1: 7 block at ages of 1 to 28 days, of curing in dry air at 70 deg. F. as compared with curing in moist air at 100 deg. F. As no tests were made on block cured for 28 days in moist air at 100 deg. F. the

average values shown in Fig. 2 for the 28-day compressive strengths of block kept at 70 deg. F. and sprinkled once daily for 7 days after moist air curing at 100 deg. F., were used. Results of other tests studied by the committee indicate that the values used represent closely the strengths to be expected from block cured for 28 days in moist air at 100 deg. F.

For each mix and condition of curing the strength increased with age. The block cured in moist air at 100 deg. F. showed higher strengths

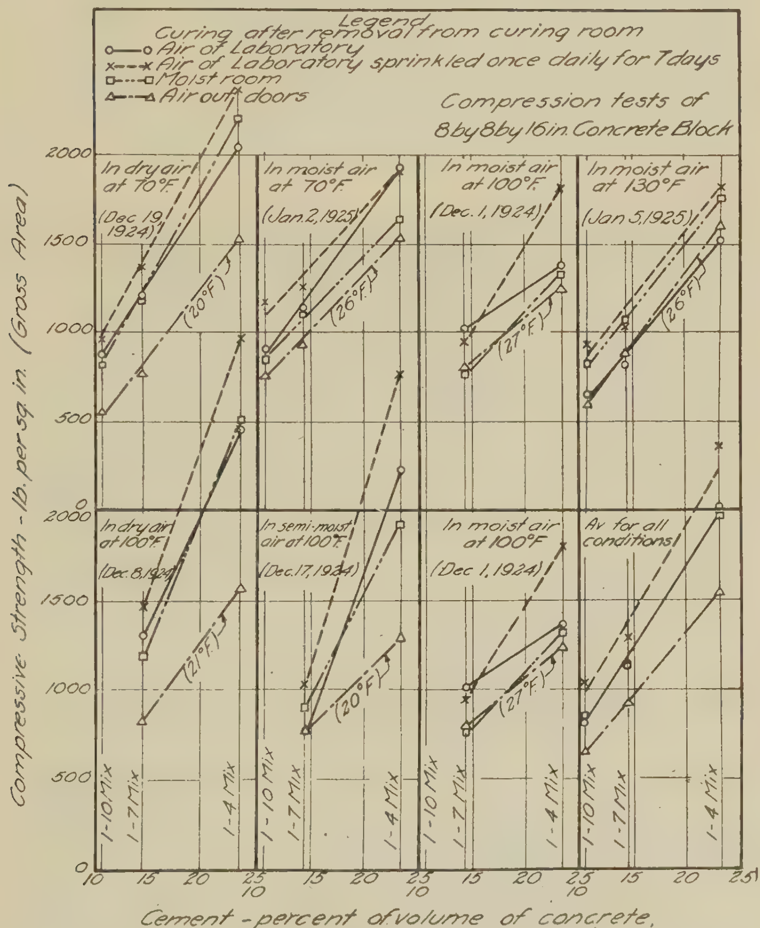

FIG. 4.—EFFECT OF QUANTITY OF CEMENT ON THE STRENGTH OF CONCRETE BLOCK AS INFLUENCED BY CURING CONDITION.

Compressive tests of 8 x 8 x 16-in. block. Age at test—28 days. Mix by volume. Data from Table 4.

at all ages. The difference in strength between the two curing conditions was greater for the 1: 7 than for the 1: 4 mix. The 1: 7 block cured in dry air at 70 deg. F. showed little gain in strength after 7 days.

Fig. 4 shows the relation between the percentage of cement per unit of volume of concrete and the 28-day strength of block cured in the curing

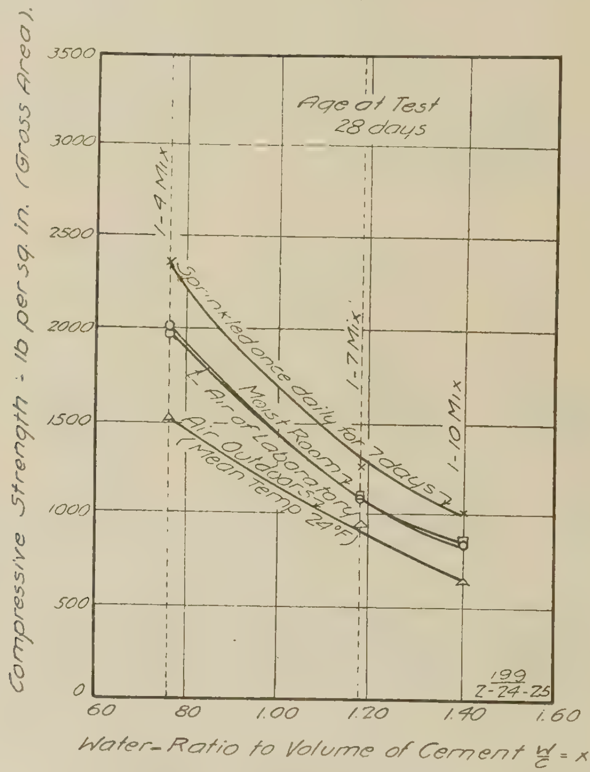

FIG. 6.—RELATIVE COMPRESSIVE STRENGTHS OF CONCRETE BLOCK CURED AT DIFFERENT TEMPERATURES.

Compressive tests of 8 x 8 x 16-in. block. Mix by volume. Data from Table 4.

room at the plant for 2 days under different moisture and air conditions and the remainder of the curing period in

(a) air of laboratory,
(b) air of laboratory sprinkled once daily for 7 days,
(c) moist room in laboratory,
(d) outdoors exposed to cold weather.

The diagrams show that in each case the strength increased with increase in amount of cement. The average increase in strength was about

1 per cent for each 1 per cent increase in cement. Sprinkling once daily for 7 days after removal from curing room generally gave somewhat higher strengths than curing in moist room at 55 deg. F. or in laboratory. It is probable that the block cured in the moist room were not completely dry when tested. Furthermore, the temperature of the moist room was generally lower than 70 deg. F. which was the temperature of the air of the laboratory. These conditions probably account for the lower strengths as compared with the block which were sprinkled. The block cured outdoors exposed to winter weather gave the lowest strengths.

Comparing the charts showing block cured in *moist air* at 70 deg., 100 deg. and 130 deg. F., it is apparent that the temperature at which the block are cured during the first seven days has no effect on the 28-day strengths provided that the curing temperature during the entire 28 days

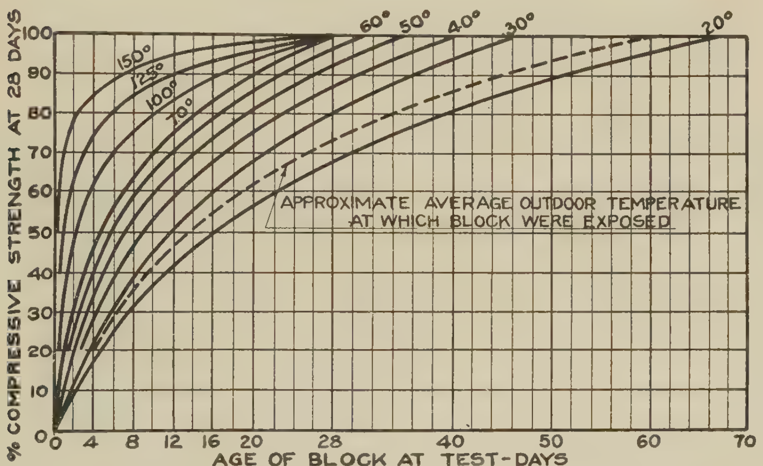

FIG. 6.—RELATIVE COMPRESSIVE STRENGTHS OF BLOCK CURED AT DIFFERENT TEMPERATURES.

Data from Table 4 these tests and Table 3 of 1924 report of the Committee. See 1924 *Proceedings*, p. 639.

is 70 deg. F. or higher (see Fig. 4). For a given temperature in the curing room, the 28-day strengths were not affected by the moisture condition.

The average 28-day strength of the 1: 10, 1: 7 and 1: 4 block for the different moisture and temperature conditions in Fig. 4 are plotted in Fig. 5. The average curves for the four curing conditions show that compressive strength decreases as proportion of the volume of mixing water to the volume of the cement increases.

The average relative values of the various curing conditions are shown in Table 5. The method of grouping in this table minimizes the variable

TABLE 4.—TESTS OF CONCRETE BUILDING BLOCK.

Compression tests of 8 x 8 x 16-in., 3-oval-core hollow concrete building block.
Gross area of block, 127.5 sq. in.; minimum area, 82.7 sq. in.
Mix by volume (based on dry and rodded aggregate).
Cement: Portland.
Aggregate (F. M. 3.80) a mixture of 0 to No. 8 sand, 0 to No. 8 limestone screenings and No. 4 to 3/8 in. crushed limestone.
Consistency, as wet as practicable.
Block made on Universal Automatic block machine at Bert Carey & Co. plant, Forest Park, Ill.
Block cured in curing room at plant for different periods thereafter under various conditions at laboratory as indicated.
Block tested at 28 days were air-dried in laboratory 3 days before test.
Loaded surfaces of block were capped with a mixture of equal parts of neat cement and gypsum.
Load was applied through a spherical bearing block placed on heavy steel distributing plate on top of specimen in order to insure an even distribution of the load.
In general, each value for strength is the average of three specimens.

Reference No.	Date Made	Age at Test, days	Mix	Curing Room: Temperature, deg. F.	Curing Room: Air Condition	Days in Curing Room	Compressive Strength, lb. per sq. in.: Gross Area: Air of Laboratory	Gross Area: Sprinkled for 7 Days	Gross Area: Outdoors	Gross Area: Moist Room	Minimum Area: Air of Laboratory	Minimum Area: Sprinkled for 7 Days	Minimum Area: Outdoors	Minimum Area: Moist Room
1–4	12–1–24	28	1:7	100	Moist	1	1,410	1,520	1,230	1,420	2,170	2,350	1,900	2,180
5–8	"	"	"	"	"	2	1,510	1,460	1,310	1,270	2,320	2,250	2,010	1,960
9–12	"	"	"	"	"	3	1,370	1,460	1,150	1,230	2,120	2,250	1,790	1,890
13–16	"	"	"	"	"	7	1,310	1,690	1,120	1,140	2,010	2,610	1,720	1,750
17–20	"	"	1:4	"	"	1	1,680	2,100	1,580	1,650	2,580	3,270	2,480	2,580
21–24	"	"	"	"	"	2	1,870	2,300	1,740	1,820	2,890	3,550	2,680	2,790
25–28	"	"	"	"	"	3	2,100	2,010	1,570	1,560	3,230	3,150	2,460	2,400
29–32	"	"	"	"	"	7	1,980	2,090	1,730	1,800	3,050	3,220	2,670	2,700
33	"	1	1:7	"	"	1	430				660	....	....	....
34	"	2	"	"	"	2	730				1,120	....	....	....
35	"	3	"	"	"	3	850	Block tested same day received.			1,300	....	....	....
36	"	7	"	"	"	7	1,060				1,630	....	....	....
37	"	1	1:4	"	"	1	720				1,100	....	....	....
38	"	2	"	"	"	2	1,150				1,780	....	....	....
39	"	3	"	"	"	3	1,180				1,820	....	....	....
40	"	7	"	"	"	7	1,480				2,280	....	....	....
49–52	12–17–24	28	1:7	"	Semi-moist	2	790	1,020	790	900	1,230	1,570	1,220	1,390
53–56	"	"	1:4	"	"	2	2,230	2,790*	1,290	1,920	3,430	4,290	1,990	2,960
41–44	12–8–24	"	1:7	"	Dry	2	1,300	1,480	850	1,190	2,000	2,280	1,310	1,840
45–48	"	"	1:4	"	"	2	2,470*	2,950*	1,570	2,510*	3,810	4,540	2,420	3,870
119–122	1–2–25	"	1:10	70	Moist	2	910	1,180	770	840	1,400	1,820	1,190	1,300
73–76	"	"	1:7	"	"	2	1,140	1,260	950	1,110	1,760	1,940	1,470	1,710
77–80	"	"	1:4	"	"	2	1,940	1,930	1,550	1,650	2,600	2,990	2,390	2,540
115–118	12–19–24	"	1:10	"	Dry	2	870	950	580	820	1,350	1,480	890	1,260
57–60	"	"	1:7	"	"	2	1,200	1,370	790	1,190	1,860	2,110	1,220	1,840
61–64	"	"	1:4	"	"	2	2,060	2,370	1,550	2,200	3,180	3,650	2,380	3,390
105	12–8–24	1	1:7	"	"	1	330	....	....	....	510	....	....	....
106	"	2	"	"	"	2	460	....	....	....	720	....	....	....
107	"	3	"	"	"	2	660	....	....	....	1,020	....	....	....
108	"	7	"	"	"	2	960	....	....	....	1,470	....	....	....
109	"	28	"	"	"	2	1,100	....	....	....	1,700	....	....	....
110	"	1	1:4	"	"	1	620†	....	....	....	960†	....	....	....
111	"	2	"	"	"	2	990†	....	....	....	1,530†	....	....	....
112	...	...	...	...	...	..	(No 3 day tests; plant broke down)					....	....	....
113	"	7	"	"	"	2	1,810†	....	....	....	2,780†	....	....	....
114	"	28	"	"	"	2	1,970‡	....	....	....	3,040‡	....	....	....
123–126	1–5–25	"	1:10	130	Moist	2	660	930	610	850	1,020	1,430	950	1,230
97–100	"	"	1:7	"	"	2	840	1,050	900	1,070	1,290	1,620	1,380	1,640
101–104	"	"	1:4	"	"	2	1,530	1,830	1,610	1,770	2,350	2,830	2,480	2,730

* Block loaded to capacity of 300,000-lb. machine, then broken in half and the half block tested. Values are average of the half blocks.
† Average of two specimens.
‡ One specimen only.

due to the making of block on different days. It will be noted that the relative values for "moist room" and "indoors at 70 deg. F. room humidity" are quite uniform. Strengths for outdoor curing are less uniform because of the wide variation in outdoor temperatures.

The relative strengths at various ages compared to the 28-day strength of block cured in moist air at 100 deg. F. and afterwards stored at 70 deg. F. and sprinkled for 7 days, is shown in Fig. 6. The curves are based

TABLE 5.—SUMMARY OF CURING TESTS.

Compressive tests of 8 x 8 x 16-in. concrete building block.
Age at test, 28 days.
Date from Table 4.
All blocks were dried at room temperature three days before testing at 28 days.
Strength ratios are the average ratios of the 28-day strengths of blocks stored under a given condition to the strength of similar blocks stored to 70° F. and sprinkled the first 7 days after initial curing.

Initial Curing Conditions	Average Strength Ratios, per cent				Number of Blocks Tested
	In Laboratory at 70° F., sprinkled 7 days	Moist Room at Laboratory until 3 days before test	In Laboratory at 70° F.,; not sprinkled	Outdoors at about 24° F.; inside 3 days before test	
Moist Air; all temperatures and mixes	100	85	91	78	144
Medium Moist Air; all temperatures and mixes	100	78	97	62	24
Dry Air; all temperatures and mixes.	100	86	87	58	48
70° F.; all moisture conditions and mixes	100	85	88	69	60
100° F.; all moisture conditions and mixes	100	82	88	72	115
130° F.; moist air and all mixes	100	97	78	98	36
1:10 mix; all moisture and temperature conditions	100	80	76	66	36
1:7 mix; all moisture and temperature conditions	100	86	88	74	111
1:4 mix; all moisture and temperature conditions	100	84	88	72	109
All conditions of moisture and temperature for all mixes	100	82	88	72	232

on these tests supplemented by all of the data in Table 3, of the 1924 report of the committee.

These curves in Fig. 6 show the effect of the temperature at which block are cured on the rate of gain in strength. Although early strengths are increased by temperatures above 70 deg. F. there was no increase at 28 days. Reduction in strength due to curing at temperatures below 70 deg. F. can be overcome by storing more than 28 days.

The relative strengths of block cured in moist air up to 28 days at 70 deg. and 100 deg. F. and for the various conditions of subsequent storage except "in moist air," are shown in Fig. 7. Curing at temperatures above 70 deg. F. for several days as compared to one day produces a maximum difference in strength of approximately 15 per cent, which occurs about the fifth day. This is shown by Curves A and B, Fig. 7.

Block not sprinkled gained higher strengths than block sprinkled during the first week. This is due to the fact that concrete is stronger in a dry condition than in a wet condition. This is shown by Curves B and C and B′ and C′, Fig. 7.

It is evident from these curves that the 28-day strength depends on the condition of subsequent storage and not the condition of the initial curing.

Conclusions.

1. For all conditions of moisture and temperature during the initial curing period, the relative compressive strengths at 28 days for the various subsequent storage conditions are as follows:

		Per Cent
(1)	Air of laboratory sprinkled once daily for 7 days..	100
(2)	Air of laboratory	88
(3)	Moist room at laboratory	82
(4)	Outdoors exposed to cold weather	72

2. For block cured in moist air at 100 deg. F. for periods of 1 to 7 days and thereafter as in 1.

The 28-day strength was in general not increased when the initial curing period was extended to 7 days.

3. For block cured in an inclosed curing room at plant for 2 days in moist, semi-moist, or dry air at temperatures of 70 deg. to 130 deg. F. and the remainder of the curing period as in 1.

(a) The 28-day strength increased with increase in amount of cement for all conditions of test. The increase in strength averaged about 1 per cent for each 1 per cent increase in cement.

(b) Temperatures from 70 to 130 deg. F. during the 2-day initial curing period did not affect the 28-day strengths.

(c) The amount of moisture supplied to the curing room during the 2-day initial curing period did not affect the 28-day strength.

4. For all conditions of moisture and temperature during initial curing and subsequent storage it is evident that the 28-day strength of concrete block depends on the condition of subsequent storage and not on the condition of initial curing.

5. There was a well-defined relation between 28-day strength and the water-cement ratio. The strength decreased with increase in water-cement ratio.

DISCUSSION.

EDWARD GODFREY (*by letter*).—There are two features of this report that need more definite recommendations. One of these concerns heating of materials. Many cases have been reported where the heating of materials was detrimental, and yet little attention is given to these cases. Indiscriminate heating and drying of the aggregates is recommended in this report, not only with live steam but also with fires. Of course aggregates must be free from frost or ice, but the use of hot or dry materials is not proper. Such materials absorb the water that is necessary for the hardening of the cement, and they cause excessive shrinkage of the product as well as weak concrete. These facts are practically totally ignored by writers of standard literature and reports. **Mr. Godfrey.**

The other feature of the report to which I wish to call attention concerns the consistency of concrete. The requirement that blocks made in a tamp or pressure machine be made as wet as practicable is good, but the one which calls for a concrete as dry as practicable to fill the molds completely and produce acceptable surfaces, in the cast process, is the one to which exception is taken. This is on a par with the common requirement for concrete as stiff as can be handled for any and all purposes, with the sole aim of obtaining the absolute maximum of compressive strength. Advocates of this have but one argument, and they give attention to no argument which concerns other and, in fact, more important properties of concrete. Tests are made by the tens of thousands for the sole purpose of discovering the condition that will give the greatest strength of concrete, but practically no tests are made to discover the consistency and mix that will give the maximum density and resistance to the entrance of water. These properties are more important than high compressive strength, for the concrete that is not dense will not grip and protect steel. In blocks the concrete may be of maximum strength, and yet it may be so porous that water will enter and cause an unsightly appearance in the wall. Also freezing of this water in the pores of the concrete may break it up.

Some of the best cast concrete work has been made with freely flowing concrete.

Mr. Walker. STANTON WALKER.—Mr. Wilk has brought out clearly the relation between the water-cement ratio and the compressive strength of concrete building units. It should be pointed out, however, that these different water-ratios merely reflect different quantities of cement, as the consistency of the concrete and grading of the aggregate were approximately the same in each case. The water-ratio alone probably cannot be used as a basis for the design of the semi-dry mixtures used in machine-made concrete products in the way it is used for plastic mixtures. In the case of the semi-dry mixtures, it is necessary to take into account the grading of the aggregate and consistency of the concrete as, even for the same water-ratio, certain aggregate and cement combinations may give higher strengths than others.

Tests carried out in concrete products plants by the Structural Materials Research Laboratory and the Portland Cement Association on concrete building units, using a wide range in type and grading of aggregate, showed slightly different water-ratio-strength relations for the different gradings. This was undoubtedly due to the difference in the effectiveness with which the machine handled the different mixtures.

Mr. Straub. F. J. STRAUB.—The comparison between the machine-made and the laboratory-made cylinders is misleading. The laboratory-made cylinders are allowed to stand one day and are then put in the water, and the report says the drier the mix was, the stronger it was. That is the reason, because you submerged it in water. I think the time in the machine as well as the time in the laboratory and in the water increases the strength, so it looks to be misleading.

I believe that the more cement there was in the mix the more it increased its strength where it was drier, because there was more cement for the water to work on; where it was most workable it made the best job.

Mr. Lowell. MR. LOWELL.—You contend that in lean mixes there was a smaller percentage of difference than in the rich mixes?

Mr. Straub. MR. STRAUB.—Yes, because there was more dry cement in rich mix for the water to work on. It is not a fair thing at all. If I were going to make a test on two blocks and going to let the laboratory cure one of them and I the other, I would put enough water in the one to be cured in moist air so it would work the easiest, but in the other, the one that the laboratory was going to stick in the water, I would not put much water at all.

Mr. Wilk. MR. WILK.—That is true; we showed that in the thick, bulky figure, in the 1 : 5 mix, the water-cement ratios used were 1.95 and 1.09. One was therefore 10 per cent more water than the other. The less water we used in making the cylinders, the higher was the strength.

MR. STRAUB.—That was because it was cured in a different way. I believe that if you had laid it down in water like you did the block, it would have been much stronger. Mr. Straub.

MR. LOWELL.—The blocks were cured very much in the same way; they were put in a moist room. Mr. Lowell.

MR. STRAUB.—There is a big difference. I am just bringing this up here because this is the place to talk about it, not afterwards, and I believe I am right. I believe still that I am right, just as in the first place, I believe it as firmly now as when I got up. I believe the difference is due to the different methods in curing those two blocks, and that is why you made a better job. In the cylinder, I believe the machine made a better job than hand made. Mr. Straub.

MR. BOURNE.—In our Recommended Practice we recommend that not more than 10 per cent of this very fine material as determined by volume by the decantation method, pass through the 100 mesh sieve. In making the decantation test, simply shake up a sample of sand in water for a minute and allow the very fine material to settle on top. That does not mean that there is only 10 per cent in the total mixture, because, after shaking, a certain amount is contained in the water which is in the material, and a certain amount is still in the material, and if you clean the sediment off of the top and shake the sample over again, you will find there will probably be nearly a third as much deposited as the first time. If you repeat again, you will probably find that when all the fine material is washed out you really have got pretty close to fifteen per cent. Along that general line I might state that I conducted a series of concrete block tests in co-operation with Mr. Walker and some others of our staff down in Pittsburgh recently, and some of the aggregates contained rather large amounts of material less than 100 mesh. We found that this fine material did not seem to decrease the strength very much, because it seemed to add workability. The series was too short to make any general statement, but I think with that data and with the other data we have had available, the committee was very well justified in increasing the previous limit of seven per cent to ten per cent by the method prescribed. There is one other consideration, when you think of the decantation method of testing. It is entirely a volume method and the settling of the material on top is rather loose. A ten per cent amount of sediment on top really amounts to only about five per cent by weight, and so while there may be 15 per cent by volume of fine material, it is really measured in a rather loose state and only represents probably six or seven per cent by weight. Mr. Bourne.

M. C. TOBIAS.—I believe that in the report of Committee P-6 for either 1923 or 1924, there is a table showing the comparison of strength between the block cured for 28 days in a saturated atmosphere of from Mr. Tobias.

65 to 212 deg., which shows that the difference in strength at the end of 28 days, regardless of the temperature at which the block was cured, only varies in the neighborhood of 10 per cent, showing that it makes very little difference at what temperature you cure it, even for the whole 28 days.

Mr. Bourne.

MR. BOURNE.—Unfortunately I have not the test data here, but in a recent series of tests at Pittsburgh, I got one rather interesting result in some curing experiments. It was intended to show the difference between one and two days steam curing and just air curing. The blocks that were to be air cured were put out in the yard and rather unfortunately for the comparison, it rained the next day after they were stored there and rained for three days, so you really had a comparison between good curing conditions outside and in the curing room. The average temperature of the curing room was about 90 deg. The average temperature outside was 65 deg. There was only a difference of 25 deg. between the two conditions. In the one-day test and in the two-day test, there was perhaps an increase of about 20 per cent to 25 per cent in steam curing over the outside curing. In the seven-day test, the gap had practically disappeared. Now the only story that this brief series of tests tells to me is that where there is not a greater difference than that 25 deg. between steam curing and sprinkling, it may not be economical to use artificial heat and the coal that is necessary to finish it; that and benefit from steam curing, during the summer period at least, is only gained when you maintain higher temperatures than those noted in your curing room compared to the outside temperatures. Of course, steam curing is absolutely necessary in the winter time, but it does raise a question as to the advisability of steam curing in the summer time where the outside temperature and the temperature of your curing room do not have a substantial difference.

Mr. Speckelmier.

MR. SPECKELMIER.—You brought up the point of eliminating steam curing in the summer time and what the results would be. What do you think the result would be if, in the summer time, in the manufacture of concrete blocks, they were sprinkled and left under cover for approximately twenty-four hours and then taken out, outside, re-sprinkled very thoroughly and covered? Would not that have almost the same result as steam curing, and bring up the strength more rapidly?

Mr. Bourne.

MR. BOURNE.—I think I might answer that by stating that the hardening of concrete stops immediately when it is dried out and that temperature has the effect of speeding up the hardening. Now you might consider those two features independently rather than as a combination. The advantage of your heat is that it speeds up the early hardening period, and if you get a substantial difference between the outside conditions and your curing room, you will probably get a substantial increase in the early hardening period; if the difference in the temperature is not very great. I do not think you will get a very great increase in early strength by

using steam. In other words, you have stated one particular condition, at 65 deg., under your method. If, at 65 deg. your blocks are kept moist for a week, you would probably get practically the same result as if they were kept moist by any other method at that temperature for a week.

MR. SPRECKELMIER.—If they were kept moist for seven days in the summer time and did not have any steam curing whatever how would that product compare at the end of 28 days with the product of steam curing at 100 deg. for twenty-four hours at the same period of the year, the summer time? **Mr. Speckelmier.**

MR. LOWELL.—It would be just the same. **Mr. Lowell.**

MR. BOURNE.—It would not be very far different, it would be pretty close, provided you keep it wet for a period as long as you say. That brings out another phase of curing, along the same lines that you have spoken of. Any method that will keep the product wet for a longer period of time than you have been accustomed to doing it, is undoubtedly going to help the product. There is no question about that. **Mr. Bourne.**

O. H. GOSSWEIN.—I notice on those curves that the blocks that was sprinkled for seven days in the laboratory showed the highest strength; is that right? **Mr. Gosswein.**

MR. LOWELL.—Yes sir. **Mr. Lowell.**

MR. GOSSWEIN.—Nothing has been said as to whether or not those blocks were sprinkled as they would be in commercial practice; for instance, in Spickelmier's plant, he would turn the hose on his block and the top row would get considerably more water than the lower row. I did not want the block men to get the impression that they would get the same results from sprinkling in a commercial way that you get in the laboratory, as a fog spray would be necessary to get the same results that you would get in the laboratory. **Mr. Gosswein.**

JOHN M. SIMPSON.—Not being able to put in steam on account of the possibility of having to move my whole plant before I would get any, I have been using calcium chloride, two pints to a gallon of water, to a bag of cement, and I find that I can handle the blocks and deliver them safely, by which I mean that there will be no come-back on my blocks. In the past I have refused to deliver a block under three weeks old to anybody. I think the expense of curing the blocks with calcium chloride is less than it would be to operate a steam curing plant and I wondered if the ultimate result was going to be fully as strong as steam curing. I did not know that what someone else might have had experience with it to a greater extent or for a longer period than I have had. At the present time I sprinkle these blocks and wet them down thoroughly twice a day and I get a compression of 2,830 lb. gross or 1,840 lb. net. I have not had a sand bank offered me but what I have lost the address without looking **Mr. Simpson.**

at it where I might use a bank run. I have an aggregate that I think is the best adapted for getting maximum results at a price just as cheap as screening. The aggregate I use will all pass through a ½-in. screen, and the sand from the same pit, being washed at the same time with an 8-in. head of water usually has a number of particles of sand clinging to the gravel, so that the gravel or coarser aggregate has part of the sand with it, so that in using two feet of gravel to a foot and a half of sand I get approximately about half and half. I use 1: 7 mix.

Mr. Gonnerman. MR. GONNERMAN.—Our laboratory has not made tests of concrete products containing calcium chloride but we have considerable data showing its effect on the compressive strength of 6 by 12-in. concrete cylinders. We found that at 2 days the strength of 1: 5 concrete or richer, mixed to a relatively dry consistency was from 40 to 50 per cent greater than that of similar concrete without calcium chloride. At 28 days the strength of concrete containing 2 per cent calcium chloride was about 15 per cent stronger than concrete without admixture. The maximum increase in strength was obtained with from 2 to 4 per cent of calcium chloride by weight of cement. For a 1: 7 mix, the increase in strength due to the addition of calcium chloride was practically negligible.

Mr. Simpson. MR. SIMPSON.—I never use over 2 per cent. Have you any record what the strength will be in a year or six year?

Mr. Gonnerman. MR. GONNERMAN.—We have made tests at ages of from 2 days to 3 years and with 2 per cent calcium chloride admixture the strength at 3 years was about 10 per cent higher than concrete without it. When more than about 6 per cent calcium chloride was used the strength was reduced at all ages. These results apply to mixes of 1: 5 or richer; for a 1: 7 mix all percentages of calcium chloride gave strengths lower than plain concrete after 7 days. At 3 years 1: 7 concrete with 2 per cent calcium chloride was about 10 per cent lower in strength than concrete without it.

Mr. Tobias. MR. TOBIAS.—In view of some of the facts brought out by this 1925 report on Committeee P-6 on Steam Curing, I have made a few figures and written a few notes here which take into consideration the computations made in the same report of the committeee for 1922 relative to the design of curing room equipment for a temperature of 125 deg. F. It is interesting to note what the requirements for a curing room equipment would be if the curing temperature is reduced to 70 deg. F. I have not gone into detail here in giving the heat values, but I have taken as nearly as possible the same coefficient of thermal conductivity through the walls and roof, floors and end walls as those used in the 1922 report. The total heat losses through walls, roof and floors of the five curing rooms described in the 1922 report and the heat required to keep the products in them at 70 deg. F. is 65,400 b. t. u. per hour. Assuming a boiler efficiency of 75 per cent and a heating value of anthracite coal of 13,000 b. t. u. per pound, the coal consumption per hour is 7 lb.

Using the same coefficient of thermal conductivity as used in the 1922 report for walls, roof and floor, assuming a radiator temperature of 212 deg. F. and a heat transfer of 250 b. t. u.'s per square foot or radiation surface per hour, the radiation surface required on the outside walls is 31 sq. ft. or 94 lin. ft. of 1¼-in. pipe and the radiation surface required on the inside wall is 27 sq ft. or 82 lin. ft. of 1¼-in. pipe. Adding the required square feet of radiation on inside and outside walls and dividing the cubic volume of the curing room by this sum gives one square foot of radiation surface per 49 cu. ft. of curing room.

Heating boilers are not usually rated on a horse power basis but with reference to the amount of radiation surface they will handle. For general consideration the following data may be of value in deciding on the size of boiler necessary. One pound of good anthracite coal will transmit about 10,000 B. t. u.'s to the boiler, will supply 40 sq ft. of radiation surface and will require 5 sq ft. of boiler heating surface. One square foot of boiler heating surface will supply 8 sq. ft. of radiation surface.

In selecting a boiler it is advisable that it should be of somewhat larger capacity than the calculations indicate is necessary. An oversize boiler will keep the curing rooms at higher temperature when fires are banked at night and will cut down firing during the day. Since a short lead from the boiler to the curing room is economical it is advisable to locate the boiler as near the rooms as practicable.

The system using radiation to heat the curing rooms although higher in first cost is more economical to operate and more satisfactory than the system whereby wet steam blown into the curing rooms is depended upon to furnish both heat and moisture.

The reason is that in blowing wet steam into the curing room about 1,200 B. t. u.'s of heat in each pound of steam blown in is lost, and another pound of water at about 35 or 40 degrees must be put into the boiler to replace the steam which if radiation was used to furnish heat would be returned to the boiler at a temperature of around 212 degrees. In other words, about 180 heat units in each pound of steam blown into the curing room are wasted.

MR. HILKER.—What would be the effect if you ran that live steam into a well or trough and let it vaporate the water? **Mr. Hilker.**

MR. TOBIAS.—The same result would be obtained but at greater expense. The term "live steam" used in connection with exhaust low pressure steam from a heating boiler is misleading. Such steam has very little so-called life in it. It is also sufficiently moist to be called wet steam. Air at 125 deg. F. or at 75 deg. F. will carry only a given amount of moisture. If enough moisture is given to the air to saturate it, there is no use putting in any more. Additional moisture put in will only condense and run down the walls of the curing room. **Mr. Tobias.**

Mr. Hilker.

MR. HILKER.—The condition you mention first occurred in a plant close to where you are located and they used a good, rich mix. They sent out the block in twenty-eight days and they put the building on it and the block crushed down. They sent an S O S to the association and they sent a man down there, and when he took his first batch of block out that morning, they were like chalk; that was from the superheated condition where he had his radiation too close to the ceiling and was blowing live steam into the room.

Mr. Tobias.

MR. TOBIAS.—That would depend also on the pressure you were carrying in your boiler.

Mr. Hilker.

MR. HILKER.—As far as steam curing is concerned, another installation would depend entirely on where that is located and how much it cost to install and maintain. Unfortunately, in my case I have to buy coal. We screen our coal and get half inch screenings that we don't know what to do with and we haul it to the boiler plant and run our steam line on that, and while we charge up the same price for the operation of that thing, the same as the coal cost us, yet if we did not use it in the block plant, it would be a dead loss to us.

Mr. Tobias.

MR. TOBIAS.—Anthracite coal is used in my calculations because cast iron sectional heating boilers are rated on the basis of anthracite coal. If Illinois Mine run coal or possibly slack is used it will be necessary to burn more coal to get the same amount of steam out of the boiler.

The radiation for curing rooms is composed of banks of pipe hung on the side walls with a return to the boiler thereby feeding the condensed steam back into the boiler instead of constantly feeding cold water in the boiler to replace that blown into the curing room as in the latter case. In the first system, one or two small pipes with perforations or nozzles are run the length of the curing room to furnish sufficient moisture for curing purposes. Both wet steam pipes and radiators for each curing room should have separate shut-offs located at the entrance to the room. This makes it possible to shut off the wet steam at night and to bank the fires without wasting water out of the boiler during the night, which is important, especially in small plants where no night man is on duty. In the ordinary wet steam installation, it is probably necessary to inject water in the boiler once an hour or oftener, depending upon its size.

Radiators are preferably located on side walls as near the floor as possible and still allow sufficient fall for the return to the boiler. Wet steam pipes should be located at the junction of the side walls and floor and perforations or nozzles so located as to cause the steam to percolate up through the block and to assist in air circulation.

In order to prevent moisture from curing rooms entering the working room and condensing on machinery and other equipment besides resulting in unpleasant working conditions, it is advisable to leave a ventilated passage-way between the entrance of the curing rooms and the entrance of the working room.

Some other suggestions have come to mind since preparing this discussion. One is the size of the steam main from the boiler to the individual radiators in the curing room; I think the committee in 1922 report gave it as four inches. It is not always possible to use a four-inch main, and in such a case use as large a main as the connections on the boiler will allow. One of the main things in a steam heating plant, so design that loss in head at the various radiators is as nearly equal as possible, to prevent steam flowing against condensation and causing water hammer in pipes.

J. P. Wilson.—I think one advantage in using a return system for radiation is that your return water does not scale up your boilers. Those cast-iron boilers are hard to clean if they become dirty, and by returning your water the boiler will remain clean and therefore more efficient than if you are using fresh water all the time. **Mr. Wilson.**

Howard Rhode.—It might be of interest to relate a little experience I had in the matter of economy, where a man started in the block business and could not afford to put in steam curing chambers. His product was not up to standard and he could not meet the building inspector's requirements. He sprinkled his blocks out in the yard in open storage with very little success. On an adjacent property was an ice manufacturing plant, and I arranged to run the condenser water—I do not know the temperature, I took no record of the thing and do not know how correct it would be in practice—from this overflow and sprayed these blocks continuously with this hot water. It raised his compressive strength about 45 per cent and put him in the running, and the man has been operating for about a year under that process, and now proposes to put in regular chambers. It was a matter of economical arrangement that tided him over the first year and made it possible for him to merchandise his product. **Mr. Rhode.**

J. P. Wilson.—Another experience along that line; I know a manufacturer who did not have room for steam kilns. He took a regular hot-air furnace and set it up in one corner of his building, and then ran the pipes along different parts of his shop, and in that way he was able to maintain his temperature. He was careful to keep his blocks sprinkled and he got as good results as the other fellow got with his steam. **Mr. Wilson.**

Mr. Lowell.—The main thing is not to let your product dry out for the first eight days. After that, it does not make much difference. **Mr. Lowell.**

Mr. Wilson.—And in winter, if the temperature is too low it does not make any difference whether you have any water in there or not. **Mr. Wilson.**

Mr. Wilson.—But the 700-lb. block is for a much higher type building than the others, and we do not give them much leeway. All the men making them have good plants and there is not so much excuse for their being low. All of them go into three and four story buildings and we require them to live up to the ordinance all of the time. We feel that

Mr. Wilson. our ordinance has worked very advantageously in Detroit, to both the manufacturer and the public in general and the building department. We are perfectly satisfied with the results we have had under that ordinance. The manufacturers have made a block that will go 500 lb. in twenty-eight days; most of them make a 500-lb. block in ten days, at least, so that the quality of the block is pretty good. It is so good that we have never had any trouble with damp basements and the strength of 500 lb. on the ordinary residence will give us a safety factor of about twenty. We feel it is a waste of energy and material to make any stronger block than that. The 700-lb. class is in general a tile which we allow in a bearing wall up to four stories. We feel 700 lb. is ample. We allow them to be loaded to 90 lb. and from what tests I have run in the laboratory on pier tests, I find that with piers laid up with tile with a course of brick on the outside and a header course every sixth tier that the actual bearing power of that pier is over 700 lb. per sq. in., so we feel that we are getting 100 per cent requirements out of these tile. With our load of not over 90 lb. we have got about a factor of eight, which we consider ample. We also have the same arrangement on the clay tile. They both make 700 lb. and I do not know any other city where they make the same rating. We have made about 450,000,000 brick units the past year and when the brick industry in Detroit made only 600,000,000 bricks you can see what strides the concrete industry has made in Detroit.

Mr. Tobias. MR. TOBIAS.—There is one thing I just took up with Mr. Gonnerman —to digress a little and go back to those curves, showing the effect of medium wet and dry curing conditions. I just made some figures here that show in a curing room 35 x 12 by an average of about 7 ft. high, containing 2,814 cu. ft., the air at 125 degrees F. will require only 17 pounds of water to saturate the air in the curing room. At 70 deg. that same curing room air will only hold about 3½ lb. of water. If the curing room is tight, and temperature is held constant, very little steam need be taken from boiler to maintain air saturation.

Mr. Wilson. MR. WILSON.—If you keep the block as it is when you make it, you do not need to put any more water on it. And when you are properly using a stripper machine you have more water than is absolutely necessary to bring about the chemical reaction. You also have enough water to keep the air in the kiln saturated, if you do not lose any to the outside.

Mr. Hilker. MR. HILKER.—I do not agree with this man from Detroit on that. We went into the tile business last year, did not know much about it, went at it blindly, used the same mix as we used on blocks, made some Sampson tile and gave them 24 hours' curing under steam pressure, temperature along about 70 or 80 deg. After 24 hours' curing we took them out in the yard and stacked them up. That was on the 18th of July; the sun was awful hot; there was a wind coming from the south; we got those tile out at 10 o'clock in the morning, and at 4 o'clock in the afternoon they were all chalk. We did not give them any moisture between the time we

took them out of the curing room at 10 o'clock and 4 o'clock. We worried for a day or two as to the cause. I concluded there was only half as much concrete as in the block, more air space, hot winds and a hot sun. We took the whole 1,600 and threw them against the wall and they crumbled to pieces and we made another batch and placed them in the curing room and left them there for four days. I looked after that curing myself, and on the morning of the fifth day I took one of those tiles out of the curing room and pitched it as far as I could and did not even notch the corner off of it. That is my contention for the condition of curing.

Report of Committee E-5 on Aggregates.

Committee E-5 on aggregates signifies its approval of that part of the report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, pertaining to aggregates, with the following exception. The committee believes that specific references to slag should be made in the definition for coarse aggregate by inserting the words "air-cooled blast-furnace slag" after the word gravel in the first line of paragraph 13.

The committee further recommends that the following definition be inserted in the list of definitions in Chapter II:

> *Blast-Furnace Slag:* The non-metallic product, consisting essentially of silicates and alumino silicates of lime which is developed simultaneously with iron in a blast furnace.

The committee further recommends that the following specifications for quality of slag be inserted in the specification for coarse aggregate:

> The weight of slag used in concrete and reinforced-concrete structures not subject to abrasion shall be not less than 65 pounds per cubic foot.
>
> The weight of slag in concrete and reinforced-concrete road and floor construction, and other structures subject to abrasion, shall be not less than 70 pounds per cubic foot.

R. W. Crum, *Chairman.*

DISCUSSION.

H. A. CHRISTIAN.—In the past twenty-five years we used blast-furnace slag almost exclusively in various concrete work. Our plant is down in the Lehigh Valley in Pennsylvania and some ten years ago we got some air-cooled slag from the Thomas Iron Co., in which there was a lot of iron shot, which, of course, we did not find out until the structures were completed. Now examining the structure some time afterward, there was a spalling on the surface. It amounted to from one inch to six inches in diameter and in digging the spalls off, the spalling action was shown to have been caused by the iron shot near the surface of the concrete, which oxidized—that is the iron shot oxidized, expanded and shoved away the concrete. Therefore I would suggest that in that specification, "free from iron shot" be included. **Mr. Christian**

WALTER H. WHEELER.—I think that you should also add to that specification, "slag from reverberatory furnaces," such as those used in the silver lead milling process and the copper smelting process, may be used. I have used that kind of slag with satisfactory results and no sign of disintegration after fifteen or sixteen years. **Mr. Wheeler**

CHARLES E. NICHOLS.—The battle has been waged back and forth vigorously on the subject of slag in the past, but I think a great many engineers are still very much in the air as to just what is the matter with slag to make people afraid of it. The committee, also, apparently feels so. Slag is well worthy of consideration, but for the information of other engineers, particularly those in sections of the country where it is not commonly used, I would like to suggest that the committee give the Institute in their report the benefit of all detailed information which comes to them, and which leads them to a recommendation for or against its use. It is a subject that, so far as I know, has never been covered adequately. It is more or less one of the "mysterious things" that get into concrete and a lot of engineers would like to know more about it. **Mr. Nichols**

HENRY L. HOWE.—In making laboratory determinations on the weight of slag, I think the result depends a great deal on the grading of the slag. I have not read the committee's report but have they taken that into consideration? I personally believe that 65 lb. is too light for ordinary work. **Mr. Howe**

PROF. W. K. HATT.—I would just like to call attention to the need for a test of some kind to determine the probable durability of aggregates and of building stone. There is no satisfactory test at the present time. **Mr. Hatt**

J. P. WILSON.—I would like to inquire whether, in the selection of aggregates, there has been any work done upon the expansion of aggregates due to moisture content? I have made a few experiments on stock limestone, taking a stone that will absorb probably two to two and a half **Mr. Wilson**

per cent of moisture, which will be probably the absorption of a very good concrete, and I have found that this stone itself self expands, probably two and a half times as much as the concrete matrix in which it is embedded. Whether in the alternate wetting and drying of structures this has anything to do with disintegration or not is something upon which I think some work can be done.

Report of Committee G-4, Nomenclature.

In 1919, the Committee on Nomenclature in its report included a definition of concrete. About a year later, this report was approved by letter ballot of the membership. This definition remained, with only minor changes, until 1923, when an urgent demand arose that the definition be modified so as to recognize as concrete only a material which employs portland cement as a binder. The Board of Direction asked that a discussion of the subject on the floor of the convention be held. This discussion was held during the 1924 convention, and it appeared to be the consensus of opinion of those participating in the discussion that it was not desirable to try to limit the use of the term "concrete" so as to make it refer only to concretes employing portland cement as a binder. The committee was asked, however, to reconsider the definition, with a view to determining whether any change at all should be made. The committee has held a meeting for this purpose, and voted unanimously to make no change at all in the definition. The committee endorses the reasons given in the *Proceedings* for 1923, pp. 577 to 579, for retaining the definition in its present form. The reason for not accepting the proposal to eliminate the words "Generally (always in the specifications of the American Concrete Institute)" are given in the following words, quoted from the last two sentences, p. 579, which read: " . . . the definition is better as it stands at present, and more likely to be given weight even in a court of law, because it is a careful statement. It makes the position of the Institute clear, and does not assume the prerogative of legislating outside of its own sphere." The definition unchanged is:

Concrete.—A compound of gravel, broken rock, or other aggregate, bound together by means of hydraulic cement, coal tar, asphaltum, or other cementing materials. Generally (always in the specifications of the American Concrete Institute) when a qualifying term is not used, portland cement concrete is understood.

The committee wishes to correct a statement made on p. 577 of the 1924 *Proceedings.* At the time of that report, the definition had "been a standard of the Institute" for three instead of "five" years. It had been included in a report of the committee made four years previously and republished two years and one year previously. See 1919 *Proceedings*, p. 375; 1921 *Proceedings*, p. 322, and 1923 *Proceedings*, p. 322.

W. A. Slater, *Chairman*,
Frank A. Hitchcock, *Secretary.*
John R. Lapham,
Allen B. McDaniel.

Fire Resistance of Concrete Building Units.*

Submitted by Committee P-5.

During 1924, your committee has completed its study and review of the tests carried out at the Underwriter's Laboratories and has received from the laboratories its report and copy of their recommendations based thereon. These recommendations have been submitted to the fire council of the Underwriter's Laboratories and approved by them.

Through the courtesy of the Portland Cement Association this report (a book of 100 pages) has been printed and one copy has been offered to each member of the Institute. It will be sent free of charge to every member requesting it.

The progress reports of Committee P-5 printed in the *Proceedings* of the American Concrete Institute, Vols. 19 and 20, give details of the tests. These details with further data appear in the report of the Underwriter's Laboratories; the committee, therefore, in making its final report, will not repeat data already before the members, but presents for the acceptance of the Institute as its final report, the report of the Underwriter's Laboratories above referred to.

In submitting this report your committee draws attention to several important conclusions drawn by the Underwriter's Laboratories from the observations made during the tests.

The following paragraphs summarize these observations. They are in effect conclusions with respect to the properties of hollow concrete building units in the several variables of form, composition and processes of manufacture and with respect to assemblies of these units into 8-in. walls or partitions, in so far as these properties have a bearing upon the performance of the units or assemblies of them when subjected on one side to standard fire exposure conditions:

1. *Flame Passage.*—There will be no material passage of flame, radiant heat or smoke through walls or partitions of these units while the wall or partition assembly remains in place.

2. *Spalling.*—There is practically no tendency to spall or for dislodgments from exposed areas.

3a. *Bulging.*—Bulging or distortion takes place and produces cracks in mortar joints and vertical through cracks in the units. The extent of

*Upon motion of Mr. Allen the report was accepted Feb. 26, 1925, as a final report of the committee.

this bulging is at least in part a function of the wall area. It is not sufficient to bring about collapse of a wall or dislodgments from its face when the usual limitations as to height and unbraced areas are observed.

3b. The extent of bulging in unrestrained areas was not determined. The observations appear to warrant the inference that such bulging would not exceed that of other unrestrained masonry units in like areas and thickness.

4a. *Cracks.*—Unequal expansion, the effect of temperature differences in the mass of a unit, produces vertical cracks in end shells and interior webs. The cracks appear at the approximate point of minimum thickness in these members and webs. In the case of block the location of these cracks is sufficiently far back from the exposed face to provide that the loosened shell is held in place by contact of its four edge surfaces with adjoining block in a wall assembly.

4b. The cracks appear early in the fire exposure period (within 20 minutes).

4c. For walls or partitions not subject to applied load the cracks have no material bearing upon the way in which the assembly performs as a fire retardant.

4d. In the case of loaded walls or partitions the cracks may impair the ability of the wall to continue supporting its load during prolonged fire exposure or when the assembly, including the framed-in members transmitting the load, is subject to vibration. In general, the effect of cracking is to produce two adjoining thin wall sections which are more or less effectively keyed together by the transverse members some of which may remain unbroken even after severe and prolonged fire exposure.

5a. *Heat Insulation.*—The critical temperature (300 deg. F.) on the unexposed surface of unfinished wall or partition assemblies of these hollow concrete building units not reached before 2 hr. 30 min.

5b. Temperatures in the core spaces become high enough to cause charring of exposed framed-in timbers in about one hour.

6a. *Stucco Finish.*—A finish on the exposed face of portland cement stucco, ¾ in. thick, will generally remain in place during the entire period of fire exposure. Such a finish delays the time of reaching the critical temperature (300 deg. F.) on the opposite face of the assembly for at least 30 min. The lag in temperature rise within the core spaces is approximately the same.

6b. A finish on the exposed side of ¾-in. portland cement stucco reduces somewhat the extent of cracking in end shells and interior webs but causes no appreciable delay in the elapsed time before cracking occurs.

The foregoing are general observations and conclusions, with regard to effects of fire exposure The following statements apply to variables of form, composition and processes of manufacture:

7a. *Aggregates.*—Coarse aggregates having a high siliceous content should not be used in hollow concrete units when performance in fire exposure is a consideration.*

7b. In the case of hollow concrete units where protection of structural steel is not a consideration, there appears little or no occasion for preference from the fire endurance point of view between the various other coarse aggregates.

8. *Curing and Consistency.*—No observations were made in connection with the fire endurance tests seeming to justify preference as to air vs. steam curing, or as between dry, damp or wet consistencies.

9. *Mix.*—The fire endurance test performance produced no evidence warranting preference as to cement proportion. The bond between a stucco finish and a wall assembly should be less affected by fire exposure if the "mixes" are nearly alike. In other words, when the coefficients of temperature expansion are approximately the same, a good bond will not be broken.

10. *Form.*—The data secured from the fire endurance tests do not warrant preference as to form or design of block. The results of tests on Panel P-4 suggest that face shells exceeding 2 in. in thickness would eliminate cracking in end shells and in internal webs.

Effect of Fire Hose Streams.—Application of a standard fire hose stream (see test of Panel a) immediately following a 5-hr. fire exposure will not produce openings in the wall assembly or cause spalling or dislodgments from its face. The thickness of exposed face shells will not be reduced by erosion beyond the point to which calcination has occurred.

Stability Under Impact.—The impact test of Panel G demonstrated substantial stability in restrained assemblies. The condition of block removed from other panels subjected only to the fire endurance test indicates that cracking had occurred generally in the block in Panel G. That none of the shells on the exposed face outside of the area of the blows were dislodged shows large degree stability in a restrained wall or partition even when units are generally cracked from fire exposure.

Strength Tests.—In general, the results recorded for these strength and other physical properties of block show that the samples employed in the fire exposure tests were truly representative good quality block. The results of tests made on specimen block taken from panels which had been subject to a fire endurance test of 5 hr. show a very substantial remaining margin of safety as to load bearing value in the test specimens, particularly when the usual limitations as to allowable load are considered.

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* The Fox River sand used in the manufacture of most of the block, while generally considered "calcareous sand," is shown by the chemical analysis to have a silica content of 43.76 per cent. That from the Meramec River basin had a silica content of 96 per cent. The possibility that the selection of fine aggregate having a silica content of but a nominal amount, and of coarse aggregates of similar low silica content would have avoided cracking in the block has not been investigated.

SUMMARY.

Fire endurance, fire hose stream, impact, absorption and compressive strength (before and after fire exposure) tests show: (1) That one-piece 5 x 8 x 12 or 8 x 8 x 16-in. hollow concrete building units, having cement proportions from 1 : 3 to 1 : 7 and either air or steam cured and mixed with either dry, damp or wet consistencies and of fine and coarse aggregates, of crushed limestone, of crushed slag, of crushed cinders or of sand and calcareous pebbles, when assembled into walls one unit thick and with the usual limitations as to height and unbraced areas, are capable of withstanding standard fire exposure conditions in excess of 5 hr. without serious bulging and without spalling or dislodgment and without permitting passage of flame, radiant heat or smoke through the wall or partition assembly; (2) that unloaded walls or partitions of such units may be classified as 2-hr. fire retardants, the determining point in the classification being the time when a temperature of 300 deg. F. is reached on the unexposed side; (3) that important loaded walls of these units may not be classified formally under the Standard Specifications as better than 15-min. retardants, the critical point being the development of cracks near the exposed face in end shells and in interior webs, the occurrence of this cracking affecting the stability of the wall at least as regards its continued use after the fire exposure conditions are removed; and (4) that loaded walls will perform effectively as fire retardants after shell and web cracking has occurred while they remain in place.

Walls composed of these units do not add fuel to fire. From the fire protection point of view their use is to be preferred over walls of combustible materials or of non-combustible materials the strengths of which are seriously affected by fire temperatures.

Consequent upon the above conclusions, the Underwriters' Laboratories have made the following recommendations to their fire council and these recommendations have been accepted and approved by the council.

RECOMMENDATIONS.

The following general information cards have been previously authorized and are now outstanding:

Guide 40 UC 24.4.

"Exterior Walls are of great importance in preventing the spread of exposure fires. Where the conflagration hazard exists such walls are frequently valuable in community fire protection. At critical places such walls should be without openings. Necessary openings elsewhere should be as small as practicable and protected by standard fire retardants of a rating suitable for the exposure."

Guide 40 UC 24.8.

"Interior fire stops (walls and partitions). The great value of interior walls and partitions as fire stops in safeguarding life and preventing

the horizontal spread of fire within buildings makes it essential that all walls and partitions regarded as fire stops be of materials, design and construction developing a high degree of fire resistance. When practicable such walls and partitions should be without openings. Necessary openings should be as small as possible and protected by standard retardants of a rating suitable for the exposure."

RECOMMENDATION 1.

We recommend promulgation of the following special information cards applying to exterior walls and to interior walls, partitions and vertical-shaft enclosures built-up of hollow concrete building units.

Guide 40 UC 24.4 May 1, 1924—Laboratories File R 1555

SPECIAL INFORMATION CARD.

Exterior walls of hollow concrete building units, 8-in. exterior walls, conforming to usual limitations as to loads, heights and unbraced areas, built up of standard hollow portland cement concrete building units with portland cement mortar are shown by standard fire endurance, fire hose stream, impact, compression and other tests to furnish protection against passage of flame, and dangerous temperature for the periods given in the following classifications.

The hollow units are subject to end shell and web cracking early in the fire exposure period. This cracking impairs the stability of loaded walls and of other walls of excess height and areas.

Non-load bearing panel enclosure and curtain walls.. R 2 hr.
Loaded walls .... R 2 hr.

See text in regard to stability and framed-in members.

A finish on the exposed side of portland cement stucco, ¾ in. thick, will usually furnish ½ hr. additional protection. The use of framed-in combustible load-bearing members will reduce the period of effective protection at least one-half.

See cards following Guide No. 40 UM 2.5 for manufacturers of standard hollow concrete building units.

Guide 40 UC 24.8 May 1, 1924—Laboratories File R 1555

SPECIAL INFORMATION CARD

Interior walls and partitions of hollow concrete building units, 8-in. interior walls, partitions and vertical-shaft enclosures, conforming to usual limitations as to loads, heights and unbraced areas, built-up of standard hollow portland cement building units with portland cement mortar are shown by standard fire endurance, fire hose stream, impact, compression and other tests to furnish protection against passage of flame and of dangerous temperatures for the periods given in the following classifications.

The hollow units are subject to end shell and web cracking early in fire exposure period. This cracking impairs the stability of loaded walls and of other walls of excess height and areas.

Non-loaded interior walls, partitions and vertical shaft enclosures .... 2 hr.
Loaded walls .... 2 hr.

See text in regard to stability and framed-in members.

A finish of portland cement plaster, ¾ in. thick, will usually furnish ½ hr. additional protection.

The use of framed-in combustible load-bearing members will reduce the period of effective protection at least one-half.

See cards following Guide No. 40 UM 2.5 for manufacturers of standard hollow concrete building units.

RECOMMENDATION 2.

We recommend the following as Underwriters' Laboratories Standard for hollow portland cement concrete building units.

Strength.—Hollow concrete building units shall have average strength in compression of 700 lb. per square inch of gross cross-sectional area as laid in the wall when tested not more than 28 days after manufacture. Individual units shall have a minimum compressible strength of 600 lb. per square inch of gross area.

Each factory producing standard hollow concrete units shall be provided with or have prompt access to satisfactory tests machines permitting frequent and systematic determination of the compressive strength of its product.

The procedure in making tests for compressive strength shall agree with the standard specifications of the American Concrete Institute.

In general the results of compressive strength and other tests should be fully recorded and these records should be identified and preserved for review if desired by representatives of the purchaser and user of regulatory bodies.

Dimensions.—Hollow concrete building units shall be of nominal thickness of 8 in. as laid in the wall. Their height shall not exceed 8 in. (nominal). The length shall not be less than 12 in. nor more than 16 in. (nominal). Face shells shall be 1½ in. in thickness at the thinnest point and shall average 1¾ in. in thickness.

A tolerance of ¼ in. plus or minus shall be recognized for the above dimensions but actual dimensions shall be used in computing compressive strength values.

Composition.—The cement proportions for hollow concrete building units may be either 1: 3, 1: 4, 1: 6 or 1: 7 as the manufacturer may elect provided that no change shall be made from a particular "mix" until strength and other tests have been made on specimens representative of the new proportion.

Only standard portland cement shall be used (A. S. T. M. Specification). Either a dry, damp or wet consistency may be employed provided no change is made from the established practice until strength and other tests have been made. Steam curing is preferred because uniformity is more easily secured thereby. If air curing is employed a special strength testing program may be required.

The aggregates used, both fine and coarse, including the sand, shall be secured from sources ensuring uniformity in quality and kind. Coarse aggregates of other than crushed limestone, crushed slag, crushed cinders or calcareous gravel shall not be used until their performance in fire endurance and strength tests has been determined.

The sand and aggregate proportions established for a given point of production shall be closely adhered to until compressive strength and other tests have been made of specimens representing new proportions.

Design.—One piece units with two or three square or oval cores are considered standard. Other one-piece forms may require fire endurance and other tests before they can be classed as standard.

Solid units of the composition regularly found in the hollow units of a particular plant may be furnished provided the limits as to dimensions are observed and provided further that their minimum strength in compression shall be not less than 1000 lb. per square inch as laid in the wall.

Marking.—The maker shall provide a distinctive marking in any surface of the units by means of which the plant at which it was made may be determined at least up to the time the unit is assembled into a wall or partition. Such markings shall not appear on output not intended to conform to this standard or upon portions of output shown by test to fail to comply with one or more items of this standard.

Fire Endurance Tests.—When fire endurance tests are made as specified in the foregoing items the program of the Standard Specifications for Fire Tests of Building Materials and Construction shall be followed except that panels may be not more than 6 by 6 ft. exposed areas and the fire hose stream test may be omitted if in the judgment of the testing body the data secured therefrom are not required to conform or supplement that secured in the fire endurance test. Test performance in general corresponding to that recorded for Panels H, A, G, B, D and K Underwriters' Laboratories Report R No. 1555, shall be basis of judgment as to outcome of such fire endurance tests.

RECOMMENDATION 3.

Finally, we recommend that when it has been shown that hollow (portland cement) concrete building units produced by any submittor comply with this standard (Recommendation 2) the staff may cause the promulgation of the card following form and text.

Guide No. 40 UM 2.5

Doe, John, Mfr.
Factory and/or Office Address
City and State

Hollow concrete building units 3-oval (or 2-square) cored pattern of sand crushed limestone aggregate.

STANDARD INSPECTION SERVICE.

See special information cards, Guide Nos. 40 UC 24.4 and 40 UC 25.8 for retardant classifications and for limitations applying to walls, partitions and vertical shaft enclosures built of standard hollow concrete building units.

Inspection authorities having jurisdiction should be consulted in all cases before these block are used.

Respectfully submitted,

(Signed) A. R. SMALL,
Vice-President.

The foregoing recommendations have been accepted and the action proposed therein has been taken.

UNDERWRITERS' LABORATORIES

(Signed) D. R. ANDERSON,
Secretary.

The block tested were made according to the standard practice of the American Concrete Institute. It will be noted that the above considerations apply to standard block thus made. The aggregate used were those in common use in the concrete block industry and it has been suggested that with block in which the fine aggregate was of a calcareous nature, better results under fire test might have been observed, but such fine aggregate is used in very few plants and the first duty of the committee was to investigate the fire resistance of block made according to present standard practice.

The conclusion of this program leaves a wide field open for further research. It would be of great value to the industry to secure data on the fire resistance of wall assemblies under load of two-piece units, or reinforced units and of units of different form to those tested.

The tests seem to point to the probability of greater fire resistance through increasing the thickness of the shell exposed to the fire and the Underwriters' staff have also suggested that the use of fine aggregate in which a larger proportion of calcareous material was found, might considerably reduce web cracking.

The use of such aggregate is not common as there is no record of natural sands of this nature and the other fine aggregates such as finely crushed limestone and granulated slag have not come into general use because of the difficulty in producing a workable mix with them in the ordinary concrete block machines.

Your committee feels, however, that the result of the tests already made will be of immense value to the industry and that as they become better known, they will assist in opening a way for the wider use of concrete building units.

They would urge that a new committee be appointed to carry on research work with a view to developing units and wall assemblies that give fire resistance even higher than that developed in these tests.

With the data secured from these tests, makers of individual concrete blocks can have their products tested at a much smaller expense than would otherwise be necessary, as the fund of data now accumulated is available as a basis to work from and future test programs for individual units need be far less elaborate.

This series of tests has laid a foundation for a series of investigations which could very properly cover the field of all masonry units used in wall assemblies and it may also prove a starting point for an intelligent system of inspection which will eventually extend to all mason's materials.

It is noteworthy that this is the first instance of an industry producing mason's units that has carried out at its own expense a comprehensive series of tests and published the results.

The tests would be of far greater value to the building industry if other industries producing mason's materials were also to institute similar test programs in order that an exact comparison of their fire resistance might be made.

In conclusion your committee would once again express its appreciation of the services of the Underwriters' Laboratories staff and the cooperation of so many in the concrete products industry who have contributed toward making this program successful.

LESLIE H. ALLEN, *Chairman*,
HARVEY WHIPPLE, *Secretary*.

Report of Committee S-6 on Concrete Roads and Pavements.

At the twentieth annual convention in 1924, this committee submitted proposed standards for concrete pavements embodying certain specifications for steel reinforcement. With regard to bar reinforcement, those specifications permitted only the use of structural or intermediate grade steel rolled from new billets. As a result of debate on the convention floor, the committee was instructed to consider further the matter of admitting bars from re-rolled steel and report at the 1925 Convention. The committee has the following to offer on this subject:

During 1924 the University of Wisconsin made comparative tests of new billet and re-rolled steel reinforcing bars. The results of this investigation certainly indicated that re-rolled bars are extremely unreliable in behavior. Over 40 per cent of the bars failed to meet the ductility requirements of the American Society for Testing Materials specification and 34 per cent of the specimens failed to pass the standard cold bend test. When the re-rolled bars were tested in tension after being bent, 80 per cent of them failed at the point of bend showing that in the process of bending the bar suffered serious injury at this point.

These results tend to confirm the opinion expressed by the committee at the 1924 convention.

Our conclusion is that the bars are not objectionable because they are rolled from old rails, but because this grade of steel is entirely too hard for reinforcement purposes. A uniform and reliable product is what is sought. Just as much trouble would occur from bars rolled from new billet steel if the carbon content was as high as that in rails.

Our recommendation is that the specifications stand as presented, requiring structural or intermediate grade steel rolled from new billets and complying with the requirements for these grades of the American Society for Testing Materials, serial designation A15-14.

C. R. Ege, *Secretary.*

At the American Concrete Institute Convention, February 26, 1925, a motion was adopted on recommendation of Committee S-6, to revise Standard Specifications for One-course Portland Cement Concrete Pavement for Highways in paragraph 5 b, page 696, Vol. 20, A. C. I. Proceedings excepting slag from the wear test requirement so that the last sentence will read: "Coarse aggregate, *excepting air-cooled blast-furnace slag* shall show not more than 6 per cent loss in the wear test"; and omitting from the "Note" (same page) the sentence "As a guide to the engineer, Abrams' 'Tables of Proportions and Quantities for Concrete Road Construction' are printed herewith."

A further motion was adopted approving for submission to letter ballot of Institute membership (to be canvassed before May 26), the following tentative standards:

"Two-course Portland Cement Concrete Pavement for Highways" (p. 710, Vol. 20, A. C. I. Proceedings); "One-course Portland Cement Concrete Street Pavement" (p. 716), and "Two-course Portland Cement Concrete Street Pavement."

The revision first referred to automatically affects the tentative standards and will under the rules be carried as a footnote until it has stood unamended for one year.

Recommended Practice for the Design and Construction of Concrete Dwelling Houses.

*Submitted by Committee S-5 on Reinforced-Concrete Houses.**

Foreword.

In preparing this Recommended Practice for the Construction of Concrete Dwelling Houses, the committee has recognized the natural limitations of loading and stresses in the dwelling house type. Methods of design applicable to large structures may, with safety, be simplified when applied to dwellings, and arbitrary regulation of wall thickness is justified. Certain facts of common knowledge in regard to the strength of concrete as compared with well-known types of lighter construction should be given expression in regulations for the concrete type.

These proposed regulations are not intended to discourage the use of so-called special systems, but rather to serve as a general standard for the correct use of the special systems or forms of construction as well as for those that do not class themselves as "special systems." Rules and regulations that discourage invention and progress in the adaptation of reinforced concrete to dwelling house construction are a hindrance rather than a help to the construction industry. In the opinion of your committee, ample strength will be insured for any dwelling house regardless of its type, combination of materials or arrangement of structural parts if it conforms with the general requirements as to strength and methods of design as set forth in these regulations.

No better reason for submitting these regulations need be given than the comparative newness of the use of concrete for dwellings and the protection from fire loss gained thereby as well as the need for standardization of the methods of design and procedure in preparing specifications or building regulations for concrete dwelling houses.

Recognizing that one of the duties of the Institute committees is to present information on new and useful developments in the particular field assigned to it, your Committee S-5 on Reinforced-Concrete Houses plans to submit next year data showing the present state of the art in the use of concrete and reinforced concrete for dwelling houses.

In addition to the foregoing it is anticipated that during the next year there will be prepared a final section that will include a program for

*By announcement of the Chair this report was accepted, without vote, as a progress report of the committee.

requirements under which detailed standards may be prepared for individual systems and forms of construction adaptable to concrete house construction. This section will serve as dependable and tried information for all who may have occasion to use it. It is felt that this information will confer a needed service on the building public and promote the proper and dependable use of each kind of dwelling house construction.

II. MATERIALS.

(A) Cement.

Portland Cement. 1. Only standard portland cement that fulfills the specifications of the American Society for Testing Materials for portland cement shall be used in the construction of concrete and reinforced-concrete dwelling houses.

(B) Aggregates.

General. 2. The use of aggregates in meeting the requirements of this chapter is subject to the results of tests made at a testing laboratory of recognized standing, showing that the aggregates proposed for use will give a strength factor of safety of 5 on compressive specimens at 28 days. The tests shall be conducted in accordance with the Standard Methods of Making Compression Tests of Concrete of the American Society for Testing Materials.

(C) Fine Aggregate.

General Requirements. 3. Fine aggregate shall consist of sand, crushed stone, air-cooled blast-furnace slag or other inert materials having similar characteristics. It shall be free from injurious amounts of dust, lumps of clap, soft particles or other foreign substances.

Grading. 4. Fine aggregate shall range in size from small to large, preferably within the following limits:

Passing a No. 50 Sieve: Not more than 30 per cent.
Passing a No. 4 Sieve: Not less than 95 per cent.

Impurities. 5. Fine aggregate consisting of natural sand shall not be used if it shows a darker color than the standard when tested in accordance with the Standard Method of Test for Organic Impurities in Sands for Concrete of the American Society for Testing Materials.

(D) Coarse Aggregate.

General Requirements. 6. Coarse aggregate shall consist of crushed rock or stone, gravel, crushed, air-cooled blast-furnace slag or other inert material of similar character or combinations thereof having clean, hard, durable, strong uncoated particles free from injurious amounts of soft, friable, thin or laminated pieces and from alkali, organic or other deleterious matter. Blast-furnace slag for this purpose should preferably weigh not less than 60 pounds per cubic foot.

Grading.

7. Coarse aggregate shall range in size from small to large, preferably within the following limits:

(a) For large sections or members:
Passing a No. 4 sieve: Not more than 5 per cent.
Passing a 1½-in. sieve: 95 per cent.
With no particles larger than 3 in.

(b) For sections or members less than 4 in. in the least dimension:
Passing a ½-in. sieve: 95 per cent.
With no particles larger than ¾ in.

Rubble Aggregate.

8. Rubble aggregate shall consist of clean, hard, durable stone larger than coarse aggregate and not larger than one-man stone.

(E) Mixed Aggregate.

Bank-Run Material.

9. Bank-run material may be used providing it satisfies the requirements of Paragraphs 2, 3, 5, 6 and 7, and providing it runs uniform in grading from fine to coarse throughout the deposit used.

Boiler Cinders.

10. Clean boiler cinders may be used in concrete block or in unreinforced concrete subject to compression only.

(F) Water.

General Requirements.

11. Water for concrete shall be clean and free from oil, acid, alkali, organic matter or other deleterious substances.

(G) Reinforcement.

General Requirements.

12. Metal reinforcement shall be of a quality and character meeting the requirements of the Standard Specifications of the American Society for Testing Materials for the kind or grade of reinforcement used. The areas of deformed bars shall be determined by the minimum cross-section thereof.

Cast Iron.

13. The quality of cast iron used in composite columns shall conform to the requirements of the Standard Specifications for Cast Iron Pipe and Special Castings of the American Society for Testing Materials.

III. DESIGN.

(A) Nomenclature.

14. The symbols used in the following formulas are defined as follows:

A_c = gross sectional area of concrete.
A_s = the effective cross-sectional area of metal reinforcement in tension in beams or compression in columns.
b = width of rectangular beams or width of flange of T-Beam.
d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement.

D = least horizontal dimension of column, pier or pilaster.
f_c = compressive unit stress in extreme fiber of concrete.
f_s = tensile unit stress in longitudinal reinforcement.
l = span length of beam or slab, or unsupported length of column, pier or pilaster.
M = bending moment or moment of resistance.
M_s = resisting moment of steel.
M_c = resisting moment of concrete.
o = perimeter of the bar.
P = the carrying capacity of an axially loaded column.
p = ratio of effective area of tension reinforcement to effective area of concrete beams—A_s/bd.
u = the bond stress per square inch of superficial area of the bar.
V = maximum vertical shear, or shear at the critical section.
w = uniformly distributed load per unit length of beam or slab.
t = thickness of flange of T-Beam.
n = ratio of modulus of elasticity of steel to that of concrete.

(B) Working Unit Stresses.

15. The following working unit stresses on concrete are based on a compressive strength of 2000 lb. per square inch at 28 days of the concrete cast in place: **Concrete.**

Extreme fiber stress in flexure—
650 lb. per square inch.

Extreme fiber stress compression, adjacent to support of continuous beams—
700 lb. per square inch.

Shear in concrete—
40 lb. per square inch.

Direct compression on plain concrete in piers, pedestals and footings—
500 lb. per square inch.

16. Tensile stress on billet steel and rail steel bars 16,000 lb. per square inch. **Reinforcement.**

17. The unit working stress for masonry walls, piers and pilasters built of concrete masonry units laid in portland cement mortar shall not exceed the following: **Working Stresses for Masonry Units.**

Hollow concrete block: 100 lb. per square inch of gross cross-sectional area.

Solid concrete block: 120 lb. per square inch of gross cross-sectional area.

Concrete brick: 250 lb. per square inch of gross cross-sectional area.

(C) Design Loads.

Loading Requirements.

18. All parts of the structure shall be designed to carry the dead loads and in addition the live-loads herein specified.

(a) Floors: Wood floor joists, 40-lb. beams, arches and similar members of reinforced-concrete—40 lb. per square foot of tributary area.

(b) Roofs: Roof beams, rafters, purlins, slabs, etc.—30 lb. per square foot of horizontal projection.

Where the climate is such that no snow loads occur, the roof live-loads may be taken as 20 lb. per square foot of horizontal projection.

(D) General Principles of Design.

General.

19. All parts of the structural supporting frame bearing loads or resisting stresses shall be designed so as to support the dead weight of the structure, including all permanent parts or members, and in addition all live-loads, without exceeding the allowable unit working stresses for the kind of materials, systems, units and forms of construction employed in the building.

20. Calculations and allowable unit working stresses shall, unless otherwise prescribed, conform to the standards of the Report of the Joint Committee on Concrete and Reinforced Concrete.

(E) Beams.

General.

21. Beams or slabs shall not be considered as fixed or restrained, unless fixity or restraint is positively provided by monolithic construction with supports and by the use of appropriate reinforcement. The width of the flange shall not be taken greater than three times the thickness of the web or slab and not greater than 2/3 of the spacing of beams.

Moment in Simple Span.

22. For simple beams and slabs the positive moment at center of span shall be determined by the relation:

$$M = \frac{wl^2}{8} \quad \text{.......................} \quad (1)$$

Moment in Continuous Beams.

23. For beams or slabs continuous over two spans, the positive moment at center of span shall be determined by the relation:

$$M = \frac{wl^2}{10} \quad \text{.......................} \quad (2)$$

and for negative moment over the middle support:

$$M = \frac{wl^2}{8} \quad \text{.......................} \quad (3)$$

24. The resisting moment of rectangular beams or slabs may be computed by the following approximate formulas:

Resisting moment of steel—

$$M_s = f \times \tfrac{7}{8} d \times A_s \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Resisting moment of concrete—

$$M = \frac{bd^2}{6} \times f_c \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

25. Where slabs are cast integrally with beams the floor slab and beam may be assumed to act as a T-Beam and computed by the following approximate formulas for T-Beams with neutral axis in the web: **T-Beams.**

$$M_s = A_s f_s (\dot{o} - \tfrac{1}{2} t) \ldots\ldots\ldots\ldots (6)$$

$$M_c \tfrac{1}{2} f_c b + (d - \tfrac{1}{2} t) \ldots\ldots\ldots\ldots (7)$$

26. Bond stress on bars in beams and slabs shall be computed by the following formula:

$$u = \frac{8}{7} \times \frac{V}{o} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

27. u shall not exceed 80 lb. per square inch for plain bars, nor 100 lb. per square inch for deformed bars, unless anchorage of the bars is provided by extending the bar beyond the point of zero moment by means of U-hooks having a radius of bend not less than four diameters of the bar and having a straight run of bar beyond the hook, of at least six-bar diameters. If U-hooks as above specified are used the bond stresses may be 160 lb. per square inch for plain bars and 200 lb. per square inch for deformed bars. **Bond Stress.**

28. Tension reinforcement at the ends of simple beams and freely supported continuous beams shall be anchored by carrying at least one-fourth by area of the tension reinforcement beyond the face of the support for a length of anchorage of 10 or more bar diameters. Negative reinforcement shall be carried to or beyond the point of inflection. (See Section 27, "Bond Stress.") **Anchorage.**

29. The shear v in reinforced-concrete beams may be computed by the following approximate formula: **Shear.**

$$v = \frac{8}{7} \times \frac{V}{bd} \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Shear reinforcement need not be provided if the shear as computed by Formula (9) is less than 40 lb. per square inch.

30. Shear reinforcement may be accomplished by: **Shear Reinforcement.**

(a) Vertical or inclined stirrups.

(b) Longitudinal bars bent up.

(c) Combination of stirrups and bent up longitudinal bars.

Spacing.

31. The spacing s, of shear reinforcement shall be determined by the following formulas: * For values of $\propto$ from 45 to 90 deg.:

$$s = \frac{16,000\ Av}{b\ (v - 40 \sin \propto)} \quad \ldots\ldots\ldots\ldots\ldots \quad (10)$$

For values of $\propto$ less than 45 deg.

$$s = \frac{16,000\ Av\ (\sin \propto + \cos \propto)}{b\ (v - 40)} \quad \ldots\ldots \quad (11)$$

in which

$\propto$ = angle between shear reinforcement and longitudinal tension reinforcement.

A_v = area of shear reinforcement in any one plane where spacing is desired.

v = unit shearing stress on section.

Combined Stirrups and Web Bars.

32. Where both bent up bars and stirrups are used in conjunction for web reinforcement, the spacing S_1 of points of bending up of longitudinal bars shall be determined by moment requirements. The unit shear value v, of these bars shall be determined by the formulas: For values of $\propto$ from 45 deg. to 90 deg.:

$$v_1 = \frac{16,000\ A_v}{bs_1 \sin \propto} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

and for values of $\propto$ less than 45 deg.

$$v_1 = \frac{16,000\ A_v (\sin \propto - \cos \propto)}{bs_1} \quad \ldots\ldots \quad (13)$$

(F) Flat Slabs.

Flat Slabs.

33. Flat slab floors shall be designed according to the methods for flat slab design provided in the recommendations of the Joint Committee on Concrete and Reinforced Concrete.

(G) Columns.

General.

34. Reinforced-concrete columns shall be of the type known as tied columns and shall not have a ratio of length to diameter or least cross sectional dimension less than 25. The longitudinal reinforcement shall be not less than one-half of 1 per cent nor more than 3 per cent of the column. The lateral type shall be not less than ¼ in. in diameter and spaced not more than 8 in. apart.

Formula.

35. The carrying capacity of tied columns shall be determined by the following formula:

$$P = 400\ (A_c + n\ A_s) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

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* Based on cylinder strength of concrete of 2,000 lb. per square inch.

(H) Piers and Pilasters.

36. Where the height of piers between lateral supports exceeds five times the least lateral dimension, the maximum allowable working stress shall be the reduced allowable unit stress obtained by the use of the following formula: **Reduction for Height.**

$$f' \text{ equals } f\ (1.25 - \frac{L}{20\ D}) \quad \ldots\ldots\ldots\ldots \quad (15)$$

Where

f' is the reduced allowable unit stress in pounds per square inch.
f is the allowable unit stress given for concrete block masonry.
L is the height of the pier in feet.
D is the least horizontal dimension in feet.

(I) Walls and Piers.

37. Monolithic concrete construction containing not more than 0.2 reinforcement so placed as to reinforce the structure against stresses resulting from temperature changes only shall be classed as plain concrete construction. **General.**

38. All parts of monolithic supporting walls shall be reinforced at floor and roof connections and at places subject to concentrated loads, so as to resist lateral forces and to distribute the loads to all parts of the supporting walls.

39. Solid bearing walls of cast-in-place concrete need not be reinforced except as herein required. **Solid Bearing Walls, Cast in Place.**

40. Where reinforcement is provided to assist in carrying loads and resisting stresses, the thickness of bearing walls of cast-in-place concrete construction may be less than as set forth for bearing walls, provided they shall be designed and constructed so as to carry all loads and resist all forces without exceeding the unit working stresses prescribed by the Report of the Joint Committee or prescribed by this standard as the conditions require.

41. Large precast units for construction of concrete dwelling houses shall be reinforced where necessary to prevent breakage in handling and erection. **Large Precast Unit Construction.**

42. All detail connections shall be of sufficient size and adequately reinforced to transmit all vertical and lateral forces.

43. Reinforcement in each unit shall be extended in such a manner that it may be hooked, looped or otherwise rigidly fastened to the reinforcement in contiguous units, in such a manner that the resulting structure will carry loads and resist stresses as a unit.

44. Dwelling houses constructed with structural framework supporting floors, roofs and enclosing walls shall be designed in accordance with accepted principles of engineering analysis and shall be of sufficient strength and rigidity to carry all loads and resist all lateral forces. **Structural Frame with Curtain Walls.**

45. Design computations and methods of construction not provided for herein, shall conform to the requirements of the Report of the Joint Committee on Concrete and Reinforced Concrete.

Number of Stories.

46. The number of stories in dwelling houses having concrete masonry walls of the minimum thicknesses herein permitted is limited only by the permissible height of the walls.

Height of Walls.

47. Unless otherwise provided herein, the height of any wall built of concrete masonry shall not exceed twenty times the thickness of such walls unless adequately braced by floors or roofs or by masonry piers or pilasters or buttresses or masonry cross-walls.

48. The height of walls as herein regulated shall be measured from the top of the foundation wall or from a supporting girder or from the foundation supports or other immediate support to the top of the wall under consideration.

Thickness of Foundation Walls.

49. Foundation walls built of concrete masonry units shall be at least as thick as the walls, piers, pilasters and buttresses that they support, except that walls required to be in excess of eight inches in thickness may be not less than eight inches in thickness above the slope of basement stairs for a distance of not to exceed ten feet horizontal provided the walls above are properly supported.

50. Where concrete masonry foundation walls serve as cellar or basement enclosing walls supporting not to exceed five feet of earth pressures, the minimum allowable thickness shall be eight inches. Where they serve as cellar or basement enclosing walls supporting earth pressures in excess of five feet in height, the thickness of these walls shall be increased as required for the height of the slope and the nature of the earth supported.

51. Where concrete masonry foundation walls serve as cellar or basement walls, (enclosing or otherwise) and do not resist earth pressures, they shall be at least as thick as the walls they support provided the clear story height of the cellar or basement does not exceed 9 ft. Where the clear story height of the cellar or basement exceeds 9 ft., these walls shall be at least 2 in. thicker than the walls, piers, pilasters or buttresses they support.

Thickness of Walls Above Foundations.

52. For walls more than 35 ft. in height, the minimum allowable thickness shall be 8 in. for the uppermost 14 ft. of the wall when reinforced by buttresses, pilasters, or masonry cross walls, or stud cross walls securely anchored, all of which reinforcements shall be not more than twenty feet apart, and when not so reinforced, the minimum allowable thickness shall be twelve inches, and shall be:

12 in. thick for the next lower 28 ft. of the wall,
16 in. thick for the next lower 28 ft. of the wall.

53. For walls not over 35 ft. in height, the minimum allowable thickness shall be 8 in. for the uppermost 25 ft. in height of the wall when

reinforced by pilasters, buttresses, or masonry cross walls, or stud cross walls securely anchored, all of which reinforcements shall be not more than 25 ft. apart, and when not so reinforced the minimum allowable thickness shall be twelve inches, and shall be:

12 in. thick for the next lower 15 ft. of the wall.

54. The minimum allowable thickness of the walls may be eight inches for the full height of the wall if not exceeding a height of 25 ft. to the square of the roof, or 35 ft. to the peak of the gable, provided they are reinforced by buttresses, pilasters, masonry cross walls or stud cross walls securely anchored not farther apart than 25 ft. Buildings having mansard roofs are not to be taken as coming within the regulations of this paragraph.

55. When the foregoing reinforcements are spaced closer together or further apart than 25 ft., the maximum allowable height to which the walls may be built shall be computed by the use of the following formula:

$$h^1 \text{ equals } \frac{h^2}{L} \qquad (14)$$

where:

h^1 equals the maximum allowable height of the wall reinforced as required.

h equals the height given in the foregoing.

L equals the distance apart of the required reinforcements.

56. Solid bearing walls of cast-in-place concrete may be two inches less in thickness than permitted for walls built of masonry units, but shall be not less than six inches in thickness.

57. Hollow bearing walls of cast-in-place concrete shall have a total thickness of materials of the inner and outer walls at least as great as herein required for solid bearing walls.

58. All changes in thicknesses of masonry walls may occur at the level of the floor nearest to the height specified except that on stair runs they may be carried to the underside of the stair flight above the height at which the thickness is required to change.

IV. Construction.

Hollow Bearing Walls Cast in Place.

59. Hollow bearing walls of cast-in-place concrete construction shall have the inner and outer walls securely braced together by *non-corroding* wires spaced not farther apart than twelve inches vertically and fifteen inches horizontally and properly embedded in the concrete and hooked at the ends. Other methods of bracing may be used provided they are sufficiently strong to bring the walls into common action.

60. Reinforcement shall be placed in both the inner and outer walls not less than No. 9 wire spaced not to exceed twelves inches vertically. Such reinforcement shall be placed at the mid-height of each succeeding horizontal course. Where the horizontal courses exceed twelve inches in

height the amount of reinforcement in each horizontal course shall be at least as great as here required per foot of height of walls.

61. Reinforcement shall also be placed in the walls over all openings sufficient to carry all loads and prevent cracking but not less than two-tenths of one per cent based upon one foot in vertical height of the wall.

Mortar.

62. All concrete masonry units shall be laid in portland cement mortar consisting of one part portland cement, to not more than three parts by measure of fine aggregate.

Joints and Bond.

63. All masonry units shall be laid with the vertical joints broken, and with all courses thoroughly bonded in the proper manner for the kind of unit used. All masonry facing or backing made of concrete masonry units shall be bonded to the masonry backing or facing of other masonry units with masonry bond in the proper manner for the kind of masonry units used, (the use of wall ties shall not be considered as sufficient bonding where the masonry facing is to be counted in the thickness of the wall, pier or pilaster).

Stucco.

64. Whenever the concrete masonry units are to be covered with stucco or plaster, the surfaces to receive the stucco or plaster shall be treated so as to insure the dependable attachment of the stucco or plaster.

Bearing on Walls.

65. All bearings on concrete masonry units shall be not less than four inches. Where vertical cell construction is used, the load shall be distributed by means of metal or masonry bearing plates of sufficient thickness to distribute the imposed load or the supporting course shall be filled with concrete, or other equivalent method of construction shall be used.

66. Wherever a change occurs in the thickness of walls, piers and pilasters made of concrete masonry units, the units shall be laid with a sufficient masonry bond at the top of the thicker section, and for hollow units laid with the cells vertically unless the longitudinal and cross walls of the units are properly superimposed the bearing loads shall be distributed upon the wall below by means of masonry or metal bearing loads, or the supporting course made of solid units, or solidly filled with concrete, or other equivalent method of construction may be used.

67. No wood shall be built into or made a part of any bearing wall made of concrete masonry units except for joist or beam bearing blocks or nailing strips not over 1 in. in width by the thickness of the mortar joint, provided, however, that wood bearing plates may be built into such walls for the purpose of supporting the roof rafters or ceiling joists when the distance from the same to the top of the masonry walls does not exceed 3 feet.

Waterproofing.

68. All concrete masonry foundation walls of hollow units shall be thoroughly waterproofed on the outside or else made of waterproofed units and laid in waterproof mortar, or else the excess ground water shall be carried away from the foundation walls by drains.

V. HOLLOW OR SOLID CONCRETE BLOCKS OR TILE FOR MASONRY PIERS AND LOAD BEARING AND EXTERIOR WALLS.

69. Hollow or solid concrete block or tile used for masonry piers and load-bearing and exterior walls shall be made of portland cement and fine and coarse aggregate meeting the requirements of II. MATERIALS, and must be sound, well seasoned and of a good merchantable quality. **General.**

(A) Approval and Rejection.

70. All units of each make or brand of concrete block or tile used for masonry piers and load-bearing and exterior walls must be marked or branded. All such marks or brands together with test reports thereon as herein required must be kept on record by the official having jurisdiction. Application must be made and approval issued for each such make or brand before same may be used in building construction. **Branding.**

71. Whenever in the opinion of the official having jurisdiction, concrete block does not conform to the requirements herein contained, he shall have the power to require tests to be conducted as herein provided. Concrete block which does not conform to the requirements herein contained shall not be used in building construction. **Tests: When Required.**

(B) Manufacture.

72. Materials for concrete blocks or tile used for the foregoing purposes shall be proportioned so as to obtain the maximum strength. Concrete for this purpose shall be mixed in the proportion of one part of cement to not more than seven and one-half parts of fine and coarse aggregate, measured separately by volumes. **Proportioning.**

73. Hydrated lime or well slaked lime putty may be added to the foregoing mixture in the proportion of not more than fifteen per cent of the cement used measured by volumes. **Hydrated Lime.**

(C) Individual Test Requirements.

74. The ultimate compressive strength in pounds per square inch of gross area of individual blocks or tile used for the purpose of masonry piers and interior and exterior load-bearing walls shall be not less than the following: **Strength Tests.**

	Ultimate Compressive Strength	
	Average of Three Tests	Minimum for Individual Tests
Hollow block or tile	700	600
Solid block	1000	900

(D) Test Requirements for Built-up Units.

Strength Tests.

75. The ultimate compressive strength in pounds per square inch of gross area of built-up units of concrete blocks or tile used for the purposes of masonry piers and interior and exterior load-bearing walls shall be not less than sixty-five per cent. of the ultimate compressive strength of gross area of the individual units.

(E) Absorption Tests.

Exposed to Weather.

76. When one or more faces of the concrete blocks are exposed to the weather, the average absorption of fire test samples of such concrete blocks, when dried to constant weight at a temperature of 212 to 250 deg. F., and then placed in clean water at approximately 70 degrees temperature for a period of 24 hours, removed and the surplus water wiped off, and the sample re-weighed, shall not exceed those given in the following tables:

Approvals shall be based upon one or the other or a combination of the following standards:

When the whole block is immersed and when the concrete used in the block weighs 140 or more pounds per cubic ft.	10 per cent of the dry weight of the block.
When the whole block is immersed and when the block weighs less than 140 pounds per cubic ft.	The per cent shall be 140 multiplied by 10, and divided by the dry unit weight of the concrete.
When the block is placed in water in such a manner that the weather face is fully submerged to a depth of one-half of one inch	3 per cent of the dry weight of the block.

Not Exposed to Weather.

77. When the blocks are not exposed to the weather and when the blocks will be covered on the weather face with stucco, the absorption tests are not required.

DISCUSSION.

EDWARD GODFREY (*By Letter*).—This report, purporting to follow the Joint Committee report, falls short of its requirements in the most vital element of any framed structure, namely, the columns. The Joint Committee gives some recognition to the fact that a long column is weaker and less reliable than a short column. The report of Committee S-5 allows the same unit stress on columns up to 25 diameters. This is something like 33 per cent more load than the Joint Committee would allow at this ratio. **Mr. Godfrey.**

No engineer, either inside or outside of the Joint Committee, has ever come out with a reasonable defense of the rodded column (called tied column by Committee S-5). A column of 25 diameters, as allowed by Committee S-5, if loaded to its capacity, would be exceedingly dangerous in a building. To visualize this, imagine a column 6 in. square and 12½ ft. high, loaded to 16,000 lb. With a 2 x 4-in timber as a ram one could doubtless bring down this column with a well-directed blow, and the whole building with it, if the concrete were perfect. If the concrete were not perfect, it would not be necessary to use the timber. This is the way more than a score of buildings have collapsed.

The allowances for shear reinforcement in this report are the three standards of all standard works and specifications, and no man has arisen who can stand up and defend any one of these standards. Not one of these standards requires a rod or wire of any kind whatever reaching to the support, where all shear must be carried. The beams of these standards are self-contained. Any one of them can fall away from the supports without disturbing or even stressing the so-called shear reinforcement.

Mr. Slater. W. A. SLATER.—I think there is an inconsistency between two sections here which ought to be pointed out. In Sec. II, 2, we have the statement, "the use of aggregates in meeting the requirements of this chapter is subject to the results of tests made at a testing laboratory of recognized standing, showing that the aggregates proposed for use will give a strength factor of safety of 5 on compressive speciments at 28 days." Then in Sec. III, 15, among the unit stresses given are, extreme fiber stress in flexure, 650 lb. per square inch, and 700 lb. per square inch for two different cases, both with concrete having a strength of 2,000 lb. per sq. in. The comparison I wish to make is of those working stresses with the requirement of a factor of safety of 5 on compressive specimens. The section on aggregates does not say that the mix used in these specimens is to be the same as those in the concrete used for the structure. If they are not, I do not see that Sec. 2 would have any point; therefore I assume that the mixture would be the same. If the mix is the same and you allow a working stress of 700 lb. per square inch on 2,000-lb. concrete, you certainly should hardly expect a factor of safety of 5 on those working stresses when you test the cylinders. Five times 700 would be 3,500 lb. per square inch that you would expect the concrete to carry in order to be used in these specifications.

Again, in III, 21, the requirement is made that "The width of the flange shall not be taken greater than three times the thickness of the web or slab and not greater than two-thirds of the spacing of beams." I merely call attention to the fact that that is a pretty severe specification; it does not allow as much effective width there as almost every other specification in the country now permits, and tests have justified the present specification, so I believe that that is rather more conservative than is necessary.

In III, 32, in the specification for web reinforcement, no upper limit is given to the spacing of the stirrups. The formula for the spacing of stirrups is given in terms of their sectional area, but there is nothing here, so far as I have been able to find, that would prevent putting the stirrups two or three times the depth of the beam apart. I do not imagine many designers would try to do that, but certainly there is an upper limit beyond which we should not go, and it seems to me that such a limit should be stated in the specifications.

Mr. Irwin. A. C. IRWIN.—I am very glad Mr. Ferguson referred to the reason why this committee has not followed and probably will not follow exactly the recommendations of Committee E-1. Committee E-1 is evolving a set of design regulations that are complete and, in spite of what Mr. Godfrey says, give up-to-date and progressive methods of design. But Committee E-1 is preparing its building regulations for an entirely different animal than the committee on concrete houses is thinking about. This same question came before the convention last year and I found it necessary to make these same remarks. If they have to be repeated again, the objectors

have the blame upon their own heads. We are preparing a set of simple specifications or rules for the design of simple construction that goes into a dwelling house, not a factory, not a warehouse, not a loft building or office building or hotel, but a simple dwelling house, and in approaching that problem from the practical standpoint, you must always keep that point in view. That answers at once why the unit stresses are taken as lower than those recommended by the Joint Committee. It was done purposely, intentionally, nothing was forgotten. We believe that those are the unit stresses which should be used. If they need some revision, all right, the committee is perfectly willing to listen to any argument one way or the other, but the committee believes that in the hands of the men who for the next five or ten years will be building concrete houses, those are the unit stresses that should be used. Mr. Irwin.

The same thing applies to Mr. Slater's objection to the width of the flange of the T-beam. It is conservative, admittedly; we want it so; that is why it was put there. In regard to the spacing of stirrups, the rules specifically call for the use of reinforcement against shear when the stress in the concrete exceeds 40 lb. Now that takes care of any situation so far as the spacing of stirrups is concerned. If the shear does not exceed 40 lb., you put no stirrups in at all. Now, let us take up Mr. Hollister's objection for a minute; he says there is no limitation on the size of columns. I wonder if he read this report? Sec. 34 says, "reinforced-concrete columns shall be of the type known as tied columns and shall not have a ratio of length to diameter or least cross sectional dimensions less than 25." That ties up the minimum size of the column.

F. R. McMILLAN.—This discussion has raised a point that comes up frequently—this matter of 650 or 800 lb. per square inch allowable stress. I wish someone would work out the relative cost of two dwelling houses, one in which the concrete was designed on the basis of 800 lb. per square inch and the other in which it was designed on the basis of 650 lb. per square inch. We have heard a great deal of criticism of the Joint Committee specifications and other reports, much of which hinges upon matters that have no appreciable effect on the ultimate cost of buildings. The cost of building a dwelling house, for example, is as much the cost of plant operation or plumbing, as it is of the cost of structural materials. Adding a few yards of concrete to the building adds very little to the cost. I think we lose the proper perspective in a lot of these things. Mr. McMillan

PROF. SLATER.—I am not quite satisfied with the statement regarding the spacing of stirrups. It has been stated that it is desirable to be conservative; let us grant for the present that it is; I do not think that the specification really does limit the spacing of stirrups, and that is not a step in the direction of conservatism. The formula for the spacing has in it a term representing the sectional area of the stirrup, and all you have Prof. Slater

Prof. Slater. to do to get a large spacing is to put in a large stirrup. You could put them as far apart as you wanted to, if you make your stirrup big enough, so far as the report is concerned.

Then again, on the question of the disagreement between the two sections first pointed out; granting for the present that it may be desirable to work to stresses as low as 650 and 700 lb. per square inch in compression in the beam, I do not see yet that these working stresses based upon a concrete strength of 2,000 lb. per sq. in. have been reconciled with the requirement from laboratory tests of the concrete of 3,500 lb. per sq. in.

Report of Committee C-3 on Treatment of Concrete Surfaces.

Your committee was instructed last year to report on the status of the old stucco specifications of 1914, 1915, 1919 and 1920, and the specifications for scrubbed concrete surfaces of 1911. In view of the fact that the existing Standard Recommended Practice for Portland Cement Stucco, A. C. I. adopted April 1, 1923 (C-3A-23) supersedes the old specifications, the latter are now obsolete and the committee recommends their withdrawal.

The old specifications for scubbed concrete surfaces were reviewed, and their substance has been entirely embodied in the Revised Tentative Recommended Practice for Treatment of Exterior Surfaces of Industrial Reinforced-Concrete Buildings. The old specifications are therefore obsolete and the committee recommends their withdrawal.

The committee also presents a brief report from its Subcommittee I on Stucco, a report from its Subcommittee IV on Interiors of Buildings, and a revision of the Tentative Standard Recommended Practice for Treatment of Exterior Surfaces of Industrial Reinforced-Concrete Buildings, with a recommendation that the latter be adopted as a tentative standard pending its final revision.

REPORT OF SUBCOMMITTEE I ON STUCCO.

It has been suggested that Paragraph 10 in the Standard Recommended Practice for Portland Cement Stucco give some broader expression as to the size of opening in expanded metal or wire lath and wire fabrics. The present recommendations call for expanded metal lath weighing at least 3.4 lb. per square yard, or wire lath not lighter than 19 gauge. No consideration is given to the size of mesh opening in expanded metal lath, whereas in the case of wire lath, 2½ meshes per inch are called for. There is some evidence that forms of lath or fabric having wider openings may have certain distinct advantages, and the committee feels that a requirement for larger openings up to a maximum of 2 in., for example, might be introduced in certain types of construction with beneficial results.

The following proposed changes in the Standard Recommended Practice for Portland Cement Stucco (C-3A-23) are submitted for consideration:

III. Design.

A. Par. 11.—Remove second sentence beginning "Tile should be set in cement mortar composed of," etc. . . . to bottom of section and insert as a separate sub-paragraph.

A. Par. 12.—Insert after the word "mortar" in the second sentence of the paragraph, the words "see Paragraph 11" in parentheses.

A. Par. 14.—Same change as for Paragraph 12.

IV. CONSTRUCTION.

A. Par. 20.—Change 20 (b) to 20 (c) and insert new 20 (b) as follows:

"(b) Wire Lath. Lath should be put up with ribs at right angles to the timbers. Use 1½-in. No. 11 slating nails or 1¼-in. galvanized staples. A nail or staple should be driven in at the intersection of each rib and timber."

In the notes applying to Paragraph 20 on page 13 on the recommended practice the following paragraph is proposed for insertion:

The committee recognizes that "Several varieties of steel fabric are on the market and have been extensively used with satisfactory results. Whenever they are used, the manufacturers' recommendations should be followed."

B. Par. 30 (d).—Change first sentence to read "In back-plastered construction the backing coat should be applied after the scratch coat has hardened."

J. E. FREEMAN, *Chairman.*

REPORT OF SUB-COMMITTEE IV, ON INTERIORS OF BUILDINGS.

I. OTHER AMERICAN CONCRETE INSTITUTE STANDARDS APPLICABLE TO INTERIOR SURFACES.

A. POINTING AND SMOOTHING.

"Tentative Standard Recommended Practice for Treatment of Exterior Surfaces of Industrial Reinforced Concrete Buildings." A. C. I. Vol. XIX, 1923, II A. Also 1925 Revisions.

B. CEMENT WASHES.

Same, II C (a).

C. RUBBED FINISHES.

Same, II D.

D. TOOLED FINISHES.

Same, II E.

E. PORTLAND CEMENT PLASTER.

"Standard Recommended Practice for Portland Cement Stucco." A. C. I. Vol. XIX, 1923, III, Par. 13.

II. RECOMMENDED PRACTICE FOR TREATMENT OF INTERIOR CONCRETE SURFACES.

A. PLASTERING.

GENERAL REQUIREMENTS.

Preparation of Surfaces.—All concrete surfaces upon which plaster is to be applied directly to the concrete should be cleaned and roughened, wherever necessary, by hacking, wire brushing or other effective means, so as to insure a good mechanical bond for the plaster. The surface of the concrete should be free from laitance, dust, dirt, grease and loose particles. Just prior to plastering, the surface should be wetted to such a degree that water will not be rapidly absorbed from the plaster, but not to such a degree that water will remain standing on the surface when the plaster is applied. **Surface Preparation.**

Grounds.—Ground strips or spot grounds should be used for all plastering. Ground strips may be used where the wood trim will cover them, otherwise spot grounds should be used. Door bucks and window frames may be used as ground strips if of the proper thickness and location. The grounds should be ¼ in. thick for ceiling work, ⅝ in. thick for walls, and 25/32 in. where furring is used. **Grounds.**

Use of Furring.—All surfaces of concrete, the opposite side of which is exposed to the weather should preferably be furred for the best grade of plastering, except in localities where the climate is both dry and warm. All furred surfaces should be three-coat work. **Furring.**

Waterproof Coatings.—Waterproof coatings which are especially designed to provide an adequate key for the plaster and properly seal the surface may be used instead of furring for the medium grade of plastering. **Coatings.**

Surfaces below Grade.—Lime or cement plaster should be used on all surfaces below grade, which are not thoroughly waterproofed. **Below Grade.**

Quicklime.—Quicklime should meet the requirements of the tentative specifications of the American Society for Testing Materials for quicklime for structural purposes, A. S. T. M. Serial Designation C5-24T. **Materials.**

Hydrated Lime.—Hydrated lime should meet the requirements of the Standard Specification of the American Society for Testing Materials for hydrated lime for structural purposes, (C6-24). **Requirements.**

Gypsum for Scratch and Brown Coats should meet the requirements of Section II, Gypsum Neat Plaster, of the A. S. T. M. standard specification for gypsum plasters, Serial Designation C28-21.

Gypsum for Finish Coat should meet the requirements of Section IV, Calcined Gypsum for Finish Coat, of the A. S. T. M. Standard Specification for Gypsum Plasters Serial Designation C28-21.

Sand for all plaster should approach as closely as practicable to the composition and grading given in the A. S. T. M. tentative specification for gypsum plastering sand, (C35-24T).

Water should be sufficiently free from salts and organic matter to be considered potable.

Hair or Fibre should be reasonably free from dust and from balls or knots of hair or fibre. The individual hairs or fibres should be from ½ to 2 in. long.

PREPARATION OF MATERIALS.

Materials. Preparation.

Quicklime should be slaked immediately upon receipt with enough water to make a cream. This should be passed through a No. 10 sieve and stored, with reasonable care to prevent excessive evaporation of water, for at least two weeks prior to use. If the lime is to be used for scratch or brown coat, the sand should be added to the cream of lime immediately after it is run through the sieve. The hair should not be added until the putty has become cold.

Finishing Hydrate for finish coat should be mixed with water to form a putty and stored, with reasonable care to prevent excessive evaporation of the water, for at least 24 hours prior to use.

PROPORTIONS AND APPLICATION OF PLASTERS.

Proportions and Application.

Lime or Gypsum Plaster Directly on Walls.—This should be two-coat work with ⅝-in. grounds. The two coats should be the brown and finish coat; the finish coat may be either white or sanded as specified.

Ceilings.

Lime or Gypsum Plaster on Ceilings.—This should be two-coat work with ¼-in. grounds. The two coats should be the scratch and finish coats; the finish coat may be either white or sanded as specified.

Lime Scratch Coat.

Lime Scratch Coat.—The scratch coat should be composed of one volume of lime putty (made from either quicklime or hydrated lime) and two volumes of sand, with one bushel of hair per cubic yard of sand. If quicklime is used, it should be slaked and mixed as specified under the "Preparation of Materials." With hydrated lime, the ingredients should be mixed first dry and again wet. In either case, the wet mixture should be stored as long as possible without permitting it to dry out, and should be retempered prior to use. Apply this plaster to the prepared concrete surface, using sufficient pressure to force the plaster into the interstices of the roughened concrete. Build this coat out to the grounds and rod, darby and float it to a true even surface with sharp, straight corners and angles. Let this set until practically air dry, and apply the finish coat directly to the surface of this scratch coat.

Lime Brown Coat.

Lime Brown Coat.—The brown coat should be composed of one volume of lime putty (made from either quicklime or hydrated lime) and three

volumes of sand. If quicklime is used it should be slaked and mixed as specified under "Preparation of Materials." With hydrated lime, the ingredients should be mixed first dry and again wet. In either case, the wet mixture should be stored as long as possible without permitting it to dry out and should be retempered prior to use. Apply this plaster to the prepared concrete surface, using sufficient pressure to force the plaster into the interstices of the roughened concrete. Build this coat out to the grounds and rod, darby and float it to a true, even surface with sharp, straight corners and angles. This coat shall be well worked, but the surface left sufficiently rough to provide a good bond for the finish coat. Let set until practically air dry.

Lime White Finish Coat.—The white finish coat should be composed of two volumes of lime putty (made from either quicklime or finishing hydrated lime) to one volume of calcined gypsum. The putty should be prepared as directed for quicklime or for hydrated lime. A small amount of the putty is circled out on a board, some water is put into the circle and calcined gypsum White Finish Coat.

The whole is then mixed with a trowel, adding more water if necessary. Do not mix at one time more material than can be used in thirty minutes. Do not retemper this mixture, but start each batch with clean board and tools. Apply this plaster in a thin layer over the base coat. Watch carefully for the disappearance of the glaze, the surface becoming slightly dull. When this occurs, immediately trowel down to a smooth true finish, using considerable pressure on the trowel, and brushing the surface with water if necessary. This coat should be as thin as possible without permitting the base coat to show through. It should be smooth, true and free from trowel marks or blotches.

Lime-Sand Finish Coat.—The sand-finish coat should be composed of one volume of lime putty to two volumes of sand. The putty should be prepared as directed for quicklime or for hydrated lime. The color of the sand should be as approved by the purchaser. Just before application, one-eighth volume of calcined gypsum should be mixed thoroughly with each batch of lime and sand, adding more water if necessary. Do not mix at one time more material than can be used in thirty minutes. Do not retemper this mixture but start each batch with a clean box and tools. Apply this plaster in a thin even layer over the base coat. When partially set finish with a cork or felt float to an even uniformly granular surface. Lime-Sand Finish Coat.

Gypsum Scratch Coat.—The scratch coat should be composed of one volume of neat gypsum plaster and two volumes of sand, with one bushel of hair per cubic yard of sand. The gypsum plaster should be retarded to set in not less than 1½ nor more than 7 hours when mixed with the above proportions of the sand, hair and water with which it is to be used. The ingredients should be mixed first dry and again wet in batches of such Gypsum Scratch Coat.

size that the entire batch can be used within 1½ hours. Do not retemper this mixture, but start each batch with a clean box and tools. Apply this plaster to the prepared concrete surface using sufficient pressure to force the plaster into the interstices of the roughened concrete. Build this coat out to the grounds and rod, darby and float it to a true even surface with sharp, straight corners and angles. Let this set until practically air dry and apply the finish coat directly to the surface of this scratch coat. During the setting, the plaster should be protected from drafts, so that it will set without drying too much.

Gypsum Brown Coat.

Gypsum Brown Coat.—The brown coat should be composed of one volume of neat gypsum plaster to three volumes of sand. The gypsum plaster should be retarded to set in not less than 2 nor more than 6 hours when mixed with the above proportion of the sand and water with which it is to be used. The ingredients should be mixed first dry and then wet in batches of such size that the entire batch can be used within 2 hours. Do not retemper this mixture, but start each batch with a clean box and tools. Apply this plaster to the prepared concrete surfaces with sufficient pressure to force the plaster into the interstices of the roughened concrete. Build this coat out to the grounds and rod, darby and float it to a true even surface with sharp straight corners and angles. This coat should be well worked, but the surface left sufficiently rough to provide a good bond for the finish coat. Let it set until practically air dry but protect it from drafts at first so that it will not dry out too much before it has set.

Gypsum Finish Coat.—Should be the same as specified for the finish coats of lime plaster.

B. OIL PAINTS.

Surface Preparation.

Preparation of Surface.—If the surface is freshly made or contains any substantial percentage of free lime, there should first be applied a neutralizing liquid made by dissolving 3 lb. of zinc sulphate crystals in one gallon of water. This should be applied by a brush or spray. It will react with the free lime, forming neutral compounds (calcium sulphate and zinc hydrate) which will have no effect upon subsequently applied oil liquids. Allow a period of twenty-four hours or more for the thorough drying of the cement or concrete surface, before applying paint. This treatment should only be omitted when oils are used which are not affected by lime.

Application of Paint.

Application of Paint.—The paint may be applied by brush or spray. Where a flat or matte surface paint for interior surfaces is desired, flat wall lithopone paints* should be used. Prepared mill whites of the gloss, egg-shell or flat type, are initially and more permanently white than ordinary exterior oil paints and should be used where high illumination is desired.

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*See Federal Specifications Board Specifications No. 21 for Flat, and No. 67 for Gloss Interior Lithopone Paints.

C. COLD WATER PAINTS, CALCIMINE AND WHITEWASH.

Form Greasing.—Any excess of form oil or grease will cause improper bonding of water paints to concrete, and may cause stains to appear on the surface. On forms for such surfaces, form oil or grease should be used sparingly; soft soap is desirable in place of oil or grease. The lubricant should be applied in a thin uniform coat. **Form Greasing.**

Preparation of the Surface.—Since the same smoothness of surface will be apparent before and after applying water paints, all roughness, board marks and honeycombing which are not desired in the finished surface must be obliterated before paint is applied. See directions for "Pointing and Smoothing," "Tentative Standard Recommended Practice for Treatment of Exterior Surfaces," A. C. I., Vol. XIX. **Surface Preparation.**

Application of Paint.—The paint may be applied by brush or by spray. The porosity of the surface will determine the amount of paint to be used on a given area; if the absorption is sufficiently rapid to cause excessive thickening of the paint film, the wall should be dampened with a light spray of water before the paint is applied, or the paint should be given to the wide variation in their covering capacity, hiding power, and durability. Water paints, because of their glue or casein binders, are subject to mildew growth in damp places. Moreover, they are not washable. For these reasons, oil and varnish paints should be used whenever these features are objectionable. **Paint Application.**

Whitewashes for Use on Interior Concrete Surfaces.—Whitewashes, for which formulas are here given, must be applied thin. In fact, best results will be secured if the application is so thin that the surface to which it is applied may be easily seen through the film while it is wet. The coating will dry opaque, however, and the thin coat will give better results than a thick one. These cold water preparations can be applied easily and satisfactorily with a large brush, a high-grade brush being preferable. When using a brush, however, do not attempt to brush out the coating as is done with oil paint, but simply spread it on as evenly and quickly as possible. **Interior Whitewashes.**

Any of the mixes can be applied with a pressure spray pump or with a paint gun with entire success.

If it is possible to allow the paste or cream made from either quicklime or hydrated lime and water to stand and soak for some time before use (stirring occasionally), the results will more than compensate for the time. The wash will work more easily and will also be more durable.

A point which must be remembered when preparing lime paints is that casein or glue solutions must be cold when mixed with the lime paste, which also must be cold. Heating of casein or glue paints causes the separation of an insoluble lime soap, thus making the mixture unfit for use. When adding formaldehyde to any mix special care should be taken to see that the mix is cold. It must be added very slowly and the mixture

stirred vigorously all the time. If the formaldehyde is added too rapidly, or if the mix is not well stirred, there is danger that the casein or glue in the mix will form a jelly-like mass which cannot be brought back into solution, and the batch will be spoiled. A little care, however, is all that is required for perfect success.

I. Where the best possible mixture for any class of work is desired, the following formula is recommended:

Soak 10 lbs. of casein in about 4 gal. of water (preferably hot) until thoroughly softened (about 2 hours). Disolve 6 lb. of trisodium phosphate in about 2 gal. of water and add this solution to the casein. Allow this mixture to dissolve. Make a thick, smooth cream of 25 lb. of whiting and 50 lb. (1 sack) of hydrated lime, with about 7 gal. of water, stirring vigorously. When the two mixtures are cold, slowly add casein-phosphate solution to the lime paste, stirring constantly. To this mixture just before use slowly add 5 pt. of formaldehyde dissolved in about 3 gal. of cold water, stirring constantly and vigorously. The cold lime paste resulting from the careful slaking and screening of 38 lb. (½ bu.) of quicklime may be used instead of the hydrated lime if desired.

Caution.—Do not make up more of this formula than can be used in one day.

> NOTE.—Borax may be substituted for the trisodium phosphate if the latter is not available. The mixes are not quite as durable and weather resistant as those containing the trisodium phosphate, but will give satisfaction, however, in most cases.

II. If the surface will be continuously dry, the following non-rubbing formula will be found very satisfactory.

Dissolve 3 lb. of glue in about 2 gal. of water. Make thick cream of 50 lb. (1 sack) of hydrated lime and about 7 gal. of water or carefully slake 38 lb. (½ bu.) of quicklime, straining the soft paste through a fine screen. Add the glue solution to the lime, stirring constantly. Thin to desired consistency.

III. Where an initial yellow tinge is not objectionable, the following formula may be used:

Dissolve 12 lb. of salt and 6 oz. of powdered alum in about 4 gal. of hot water. Add 1 qt. of molasses. Make a thick cream by thoroughly mixing 50 lb. (1 sack) of hydrated lime with about 7 gal. of hot water. Add the clear solution to the lime, stirring vigorously. Thin to desired consistency.

In the foregoing formula, 38 lb. (½ bu.) of fresh quicklime may be substituted for the hydrated lime. The quicklime must be carefully slaked and screened before use.

The yellow tinge disappears within a few days, and a very white, durable coating results. This mixture is somewhat cheaper than the others.

III. Problems Requiring Further Discussion and Study.

While the foregoing recommendation for plastering on concrete surfaces represents the best present-day practice, the committee feels that there is still considerable question about the adhesion of any kind of plaster to concrete. Since for the best grade of work the committee recommends furring for exterior walls, the principal problem is that of plastering ceilings without the use of furring.

It is apparent that further research on this problem is vital.

Since the problem involves all kinds of plaster, whether applied directly on concrete or upon some binding medium, it has been suggested that the following groups should be interested to the extent of co-operating in the study of the problem:

Portland Cement Association.
American Concrete Institute.
The Gypsum Industries.
National Lime Association.
Manufacturers of Bonding Coatings.

It is hoped that this question will be thoroughly discussed and a definite plan of action determined upon.

H. Whittemore Brown, *Chairman.*

TENTATIVE STANDARD RECOMMENDED PRACTICE FOR TREATMENT OF EXTERIOR SURFACES OF INDUSTRIAL REINFORCED-CONCRETE BUILDINGS.

Submitted by Committee C-3 on Treatment of Concrete Surfaces.

(Revised 1925)

I.

SCOPE AND GENERAL REQUIREMENTS.

Serial Designation C-3B-25 T.

Scope.

1. Scope.—This recommended practice is designated to outline approved methods of treating the exterior surfaces of concrete factories, warehouses, and other industrial buildings, in such manner as to produce pleasing and durable surfaces at costs which are not inconsistent with the occupancy and surroundings of such buildings.

General Requirements.

2. General Requirements.—The good exterior appearance of a concrete building depends fundamentally on care in the building of forms for the outside beams and columns. Boards used in column forms shall always run vertically, and in beams horizontally, and shall preferably be of even width in order that the board marks be continuous throughout the length of the column beam. Sound, clean lumber of sufficient strength shall be used, and the forms properly braced in order to insure straight and true general lines. The concrete shall be carefully proportioned, mixed, and placed in order to reduce to a minimum the need for pointing, patching and other corrective treatments.

NOTE.—Industrial buildings, as a rule, are located in districts where appearance is a matter of secondary importance. The cost of any surface treatment is, therefore, of prime importance, and circumstances are usually the governing factor in the selection of the type of finish. It is a fact, however, that a reasonable expenditure is justified in making the exterior of a building attractive to the public as an advertising feature, if for no other reason. Under any consideration all concrete structures should at least have all voids pointed, and exposed wires, nails and bolts removed or cut off so as to guard against disintegration from the action of the elements. From this required minimum one may go to the costly extremes of inlaid tiles or brick and stone veneers, but such treatments are to be considered from the architectural viewpoint, and are beyond the scope of this practice, which is limited to the treatment of the concrete itself as it comes from the forms. Due to the very nature of concrete, the forms are subject to a certain amount of movement while being filled, no matter how careful the supervision or workmanship. It is, therefore, esential to guard in so far as possible against bulging and distortion of forms, as

no amount of work will correct the faulty appearance of a building unless the general lines are good. Also it is obvious that care in concreting will eliminate unnecessary expenses in finishing, and at the same time give better results. A reasonable amount of precaution to prevent the occurrence of conspicuous defects is better than the cure of such defects afterwards.

II.—CLASSIFICATION OF SURFACE TREATMENTS.

3. Classes of Treatments.—The surface treatments described herein are divided into the following five general classes:

a. Pointing and Patching (minimum requirements).
b. Correction of Column and Beam Lines, Fill Joints, etc.
c. Cement Washes and Proprietary Paints.
d. Rubbed Finishes.
e. Tooled Finishes.

The recommended methods for producing these finishes are given in detail in the following sections:

A. POINTING AND PATCHING (MINIMUM REQUIREMENTS).

4. Pointing and Patching.—All nails, wires, and bolts shall first be removed or cut back to a depth of at least one inch from the surface of the concrete, so as to provide sufficient key for the pointing mortar, and to insure against water reaching any pieces of iron or steel and causing rust spots or spalling. Bolt holes shall be filled with corks or wooden plugs of ⅛-in. greater diameter than the holes. The corks shall be driven into the holes until the head is one inch back from the surface. All defective places shall be thoroughly cleaned of dust, loose pieces, laitance and foreign particles (such as sawdust). The spot to be patched and the concrete immediately around it shall then be saturated with water. **Pointing and Patching.**

5. Mortar.—A mortar of one part cement and two parts of clean building sand shall then be forced into all parts of the cavity or defective spot, and the surface rubbed with a cork or wood float. Any mortar that may work out and lap over on the sound concrete shall be removed with a clean dry brush or piece of bagging. **Mortar.**

6. Large Patches.—If large patches of considerable depth and area occur, the mortar shall be applied in two or more coats. Each undercoat shall be scored as in plaster work, but it is not necessary or desirable for each coat to become entirely dry before applying the next.

7. Curing.—Patching should be avoided in hot sunshine or quick drying wind, unless it is feasible to protect the fresh mortar with wet burlap or canvas. **Curing.**

NOTE.—The pointing and patching treatment outlined above naturally leaves a somewhat spotty appearance, but after weathering for several months the entire surface will begin to assume a

fairly uniform color. A pointed place will usually show up as a dark spot on the body of the building unless the mortar used is somewhat lighter in color than the mortar used in the concrete. This can be corrected by using a little white cement or a light colored sand in the pointing mortar. It has become quite common practice to use a white beach sand, but the committee would always recommend a bank or dredged sand which has been screened through a sieve of 10 meshes per inch. Final rubbing of patches should be parallel to board marks of surrounding area so as to make patch as inconspicuous as possible.

It is to be noted that the treatment prescribed in this section is that which is warranted entirely from the standpoint of utility, rather than from that of appearance.

B. CORRECTION OF COLUMN AND BEAM LINES, FILL JOINTS, ETC.

Correction of Lines.

8. Correction of Lines.—This section is supplementary to Section A and prescribes additional corrective treatment, but not to the extent of eliminating board marks or bringing the surface to uniform color. The surface shall first receive the pointing and patching treatment where required, as described in paragraphs 4 to 7 inclusive.

Cutting to True Surface.

9. Cutting to True Surface.—Nail head marks, fins and other small projections shall be removed. Beams which sag or bulge, and columns which are out of plumb shall be cut to line. Fill lines shall be dressed by thoroughly cleaning the joint and then applying mortar and finishing with cork or wood float, as described in paragraph 5.

NOTE.—Good judgment must be used in deciding how far the more expensive work of truing up bad lines should be carried, as considerable money may easily be spent without materially improving the general appearance.

It has been found that pounding with a flat-headed hammer will cause small projections to crumble down to the general level the surrounding surface. This method gives a better, quicker and cheaper results than cutting with a chisel, which may have to be used on projections of considerable size.

When cleaning up and pointing around windows and miscellaneous iron particular care should be taken to cut clean sharp edges, and not allow a film of mortar to lap over on the steel, for this film will in time break away and leave a ragged edge, and may, in the case of windows, cause a leak.

If a prominent board mark has been erased by the pointing, the patch will not be so noticeable in contrast to the body of the building if the mark is ruled in again.

It is estimated that the treatment prescribed in sections A and B will require about one and one-half bags of cement per 1000 sq. ft. of surface. The cost will average between 2 and 3¢ per square foot with cement masons at 75¢ per hour and labor at 40¢ per hour. The actual cost in specific cases may vary considerably from this estimate, according to the quality of the forms-work and the concreting.

Protecting Steel.

10. Protecting Steel Near Surface.—Workmen shall be instructed to watch for steel reinforcement near the surface, and if this condition should be found special precautions shall be taken to protect such steel from rust.

NOTE.—When reinforcing steel in exterior members is not back at least one inch from the outside face of the concrete, moisture may work its way in causing it to rust. Eventually this rusting if continued will spall off the concrete and cause an unsightly appearance and may even in severe cases affect the strength of the structure. Good design should never call for such sizes of hoops, spirals, stirrups, etc. as will necessitate steel within one inch of the surface, but sometimes due to careless placing the steel will be at or near the surface. All steel should preferably be 2 in. back from all exterior surfaces.

To correct this condition cut out the concrete around the steel on all sides and if the bar is of minor importance cut it off well back from the surface. When it is not advisable to cut out the steel the recess should be cut large enough so as to bend the steel back from the surface.

If important and large bars should be encountered where it is not desirable or possible to either bend back or cut off the steel the bar should be painted with red lead and then wrapped loosely with No. 16 iron wire to insure a bond and the recess carefully pointed.

C. CEMENT WASHES AND PROPRIETARY PAINTS.

(A) CEMENT WASHES.

11. Cement Wash.—Cement washes of practically any color from white or cream to cement gray can be prepared by varying the proportions of white or gray cement and light or dark sand. Before applying the wash, all pointing and patching should be completed as indicated in Sections A and B. **Cement Wash.**

12. Mixture.—A mixture of 1 part white cement and 1 part finely-screened yellow bank sand, with 5 per cent of hydrated lime (by volume of cement) will give a serviceable color just off the white, and will serve as an example of a typical dry mixture. **Mixture.**

13. Mixing.—The cement, sand and lime in the desired proportions shall be thoroughly mixed in the dry state, the mixture shall then be slowly added to water, stirring vigorously until the consistency is that of a stiff oil paint. The dry batch shall be large enough for a full day's work, but only enough of the wash shall be prepared at least one hour. The wash shall be stirred before each application to the concrete surface, and when refilling the container all the old wash shall be cleaned out and discarded. **Mixing.**

14. Application.—The area to be coated shall first be thoroughly wet, and then a full brush coat of the wash applied. This coat shall be rubbed in with a cork float, the surface being sprinkled with a little additional water if necessary. The surface shall then be gone over with a clean, damp brush, brushing in the direction of the board marks, so as to remove all excess material, and the remaining coat shall be as thin as will permit the surface to be entirely covered. **Application.**

Joinings.

15. Joinings.—In coating adjacent areas the brush marks shall be carefully blended to avoid a line between the two areas. The sequence of the work shall be so planned that joinings occur at natural breaks in the surface.

Curing.

16. Curing.—In warm or drying weather the finished surface shall be sprinkled with water once a day for three days, and in cool, damp weather shall be sprinkled at least once within 24 hours after finishing.

NOTE.—The sand must be dry before screening not only to facilitate the screening but also to avoid the possibility of moisture in the sand causing a set in the dry mix. Best results seem to be obtained with a sieve having 18 meshes per square inch.

Whenever a cement wash is to be used, less attention need be given to the color and finish of patches as described in the note under Section A, since it is apparent that the applied coating will give the desired uniformity to the concrete surface. This coating should, however, be as thin as possible, for a thick coat will eventually craze and peel off.

A very fine appearance is obtained if the wash is rubbed in with a carborundum stone. This not only insures a better bond, by more positively forcing the material into the pores of the concrete, but at the same time grinds down any slight projection, leaving a semi-rubbed surface.

Sprinkling of the freshly coated surface is necessary, for if the wash dries before it has attained its set, it will dust off. A very practical way to sprinkle the surface is to make up two perforated pieces of pipe about four feet long, on a "T," and plug the ends. Then connect a hose to the leg of the "T" and lower the pipe down from the roof over the face of the columns and walls. Spray nozzles can also be used to advantage. The sprinkling must be gentle so as not to wash off the fresh coating.

A very effective way of insuring the bond of the wash coat is to mix 10 per cent by volume, of a 40 per cent calcium chloride solution, with the gaging water. The calcium chloride attracts moisture from the air and keeps the wash coat damp for several days, thus insuring the set of the cement before it dries out.

The cement wash will cost about 2¢ per square foot, and will require about one bag of cement per 1000 sq. ft. Adding the cost of the work specified under Sections A and B the total cost of this method of surface treatment would be about 5 cents per square foot with cement masons at 75¢ per hour and labor at 40¢ per hour, and the total material one and one-half bags of gray cement and one bag of white cement per 1000 sq. ft.

(B) CEMENT PAINTS.

Proprietary Coatings.

17. Proprietary Coatings.—Whenever proprietary cement paints or coatings are to be used, the manufacturer's directions shall be followed in their application.

NOTE.—The committee recognizes that there are many paints and coatings for concrete on the market which have been widely used and have given satisfaction. It cannot, however, give its endorsement to particular proprietary materials, owing to its own

limited facilities for conducting the exposure tests and field inspections upon which ratings should be based.

It has been noted that oil paints at times have not proven satisfactory when applied in two coats. There seems to be just enough pull or tension in two coats to cause blisters. When speaking of two coats we do not consider the priming coat of zinc sulphate which is recommended by some manufacturers as a neutralizer for free lime that may be present.

If concrete surfaces are allowed to weather several months before painting the carbonic acid gas in the air will tend to unite with the free alkali in the cement. This forms a neutral carbonate of lime which will not attack the oils in paint.

D. Rubbed Finishes.

18. Rubbed Finishes.—The rubbed finishes are obtained by grinding the surface with abrasive stones. The first rub shall be completed as soon as the forms can be removed. Faces of columns, beams and walls shall be treated within three or four days. **Rubbed Finishes.**

19. First Rub.—As soon as the forms are removed the surface shall be thoroughly wetted and then rubbed with a No. 20 carborundum stone. The rubbing shall be continued until fins, board marks, nail bead marks, and to a certain extent the irregularities between boards, are removed. The cement paste which works up in the rubbing process shall be removed by washing and brushing. **First Rub.**

20. Pointing.—Voids in the concrete shall be filled with a mortar (usually 1 : 2) composed of finely screened aggregate of the same general description as that used in the concrete. This mortar shall be thoroughly worked into the face with the carborundum stone. **Pointing.**

Note.—The best and most economical results are obtained by applying the first rub to the concrete while it is "green." Filling of the small voids with mortar as described in paragraph 20 offers no difficulties, but the operator should be warned against leaving an appreciable thickness of mortar on the face of the concrete to take up irregularities in the surface, unless special precautions are taken to insure the bond. Such precautions are particularly necessary when the surfacing is delayed until the concrete has hardened and dried out.

If the concrete cannot be given the first rub when it is still "green," board marks, nail-head marks, and small projections must first be removed as indicated in paragraph 9. After thorough wetting, the surface should receive the cement wash application as described in the first part of paragraph 14, in order to help the grinding action. (See also the second paragraph in the note following paragraph 16). The carborundum rub should then be given as prescribed above, being sure to remove all the cement wash by washing and brushing.

21. Second Rub.—The second rub shall be given near the end of the work, when the building is ready to clean down and danger of staining from other work is past. The surface shall be thoroughly set and then **Second Rub.**

gone over with a No. 24 carborundum stone. The paste which is worked up shall be removed with a wet brush or clean bagging. When dry the finished surface will resemble limestone in color and texture.

NOTE.—When using the rubbing method described in this section it is essential that *all* patching, correction of lines, etc., be done before or during the first rub. When the second rub is performed nothing but the stone and plenty of water should be used. The irregularities of color or texture which are noticeable after the first rub need not be considered, as the second rub brings the surfaces to an almost uniform color.

The cost of this process including miscellaneous pointing and first and second rub costs about six cents per square foot with masons at 75¢ per hour and labor at 40¢ per hour.

E. TOOLED FINISHES.

22. *General.*—As here used "tooled finishes" includes all finishes in which the surfaces of the concrete is roughened or removed to expose the aggregate. Wire or fibre brushes, stone dressing tools, sand blast and rotary cutters may be used for this purpose. When any of these rough surface treatments are to be used great care shall be given to the proper layout of construction joints and fill lines, and plans showing the treated areas shall be issued before any concrete is placed. The work shall be planned to eliminate as many joints as possible, particularly horizontal fill lines. If the latter are unavoidable they shall be absolutely level. As this type of finish is usually carried out in panel effects great care shall be taken to lay out the exact lines and to cut clean and sharp at the edges. The borders of the panels and such other parts of the surface as are not roughened must receive the usual pointing as described in sections A and B and also one of the wash or rubbed finishes so as to balance with the tooled areas.

Brushed Finishes.

23. *Brushed Finishes.*—Immediately after removal of forms (usually 18 hours after placing concrete) the concrete shall be scrubbed until the surface film is removed and the aggregate exposed to a uniform degree and then the surface shall be rinsed off with water and kept moist for several days.

Pointing.

24. *Pointing.*—If any pointing is required the defective spots shall be patched immediately after the scrubbing. After the patches have hardened (within 6 to 24 hours) they shall be scrubbed to the same texture as the general surface and rinsed clean and kept moist for several days.

Hard Spots.

25. *Hard Spots.*—If for any reason part of the forms are not removed before the concrete has become too hard for scrubbing, the hard portion shall be treated with sand blast or tool dressing to a texture matching the scrubbed surface. After tool dressing, the surface shall be washed a 1 to 10 solution of muriatic acid and water and then thoroughly rinsed with water to remove all traces of acid.

26. Tools.—For scrubbing the ordinary "green" surface, stiff fibre brushes and the use of plenty of water will be sufficient. If the surface is too hard wire brushes, followed by fibre brushes, may be used. **Tools.**

27. Hammered Finishes.—Surfaces to be finished with a rough texture, such as bush hammering or crandalling, shall be at least two weeks old before treatment. The work may be done by mechanically operated tools, either air, or electricity, or by hand hammering. In general, the work on any one panel or area shall be done by the same operator so as to avoid differences in texture. After the coarse aggregate is exposed to a uniform degree the roughened surface shall be brushed with a fibre brush or washed with a hose stream to remove loose particles and dust.

28. Pointing.—If any pointing or filling of bolt holes is required on a hammered surface the tooling shall precede the patching. The patching shall be done by first pointing the spot and then embedding selected pieces of large aggregate in the mortar bed so as to match the surrounding tooled area. After the patches have thoroughly hardened they shall be carefully dressed. **Pointing**

NOTE.—In the committee's opinion the effects that are possible by use of the various tooling treatments are the most pleasing of any, but at the same time are the most costly and difficult to do. These treatments must also be considered entirely from an aesthetic viewpoint as they do not add to the weather-resisting qualities of the concrete and, in fact, tend to its disintegration if the concrete is not of the very best.

The success of tooling depends upon uniformity in the results obtained and therefore special attention is called to the importance of avoiding joints, fill lines, laitance and segregation of aggregate. Careful selection of the aggregate helps in obtaining the desired uniformity and if the aggregate be selected for color its exposure by tooling will introduce pleasing effects in color variation as well as in texture.

The face forms for all exposed surfaces which are to be tooled shall be straight, smooth, evenly matched and watertight and should be so built that they can be removed without prying against, or jarring the concrete. The surface of the face forms should be coated with petroleum, or other water repellent to prevent adhesion of the concrete.

The concrete should be thoroughly mixed, wet enough to flush and be spaded against the face form until the cement cream or grout is on the outside surface. The aggregate of the concrete near the face should then be spaded forward to the surface so as to obtain a uniform mixture at the face. The placing of concrete should be continuous throughout definite stages so that the joints between different day's work appear, if possible, at some feature line, or at least be made truly straight and level.

29. Chemical Process for Tooled Finish.—Recently a product has been placed on the market which by its chemical action on concrete surfaces is designed to obtain results similar in appearance to a tooled finish. This material undoubtedly has merit, but the committee is not in a position **Tooled Finish.**

to at this time make any general statements regarding its use, or the results which may be obtained, and recommends that where it is used the manufacturerer's directions be followed. Owing to the fact that this process promises a comparatively cheap method of obtaining a tooled surface which compares favorably with the more expensive methods now prevalent, its use is certainly worth consideration, and it is hoped that the experiments now under way will be able to establish more definitely its value.

DISCUSSION.

VIRGIL L. JOHNSON.—I have come here on a special mission to find out what has been done in regard to the adhesion of plaster to concrete. I merely noticed in the paper that it should be investigated. With architects, this is an important question, for if we are having schools built of concrete and the plaster is liable to fall off, it becomes a question whether or not to eliminate concrete work in our schools. Mr. Johnson.

Millions of dollars worth of school buildings have been built in the last four years and to the cement industry it is an important item. Sometimes it so happens that the girder which occurs in the ceilings sags in the center or side and the plasterer, in order to level up his work, will put on an inch and a half of plaster. Now there has been a case where plaster in large quantities has fallen off; this is dangerous to the children in the room.

J. C. PEARSON.—I think the condition the gentleman has mentioned has been recognized by everyone who is familiar with this sort of work. The difficulty of writing a specification for plaster so that it will adhere properly to concrete has certainly been demonstrated by the fact that you cannot get anybody to commit himself very definitely on what to do. The real value in this specification is that it embodies the best known practice at the present time—the committee is frank to say, however, that more work needs to be done. I think the recommendations which are here made probably give better assurance of plaster adhering to concrete surfaces, such as girders, ceilings or walls, than anything which has been published heretofore. Mr. Pearson.

E. Y. BRAGGER.—I have had experience in plastering portland cement stucco onto an impervious portland cement smooth face, and up to the present time none has fallen off. Mr. Bragger.

MR. JOHNSON.—I want to add that there is no trouble whatever with the rusting of the suspended wire lath ceiling, as we have the lath thoroughly coated with paint. The main trouble has been with the concrete girders—that is the adhesion of the plaster to the soffit of the girder. Mr. Johnson.

JOHN G. AHLERS.—We had a meeting of Committee C-3 the other day, and Mr. Earley expressed an opinion in this meeting that the object of hair or fibers in plaster was just for the period during which the plaster was applied. There is no value to these materials after setting. I think the real way to make plaster adhere is to use successively leaner coats. Your richest coat the first coat, to give adhesion, the succeeding leaner coats will give spaces for expansion and contraction and prevent spalling. Mr. Ahlers.

Report of Committee C-5 on Measurement of and Estimating Concrete.*

During the convention of the American Concrete Institute, held in Cincinnati, during January, 1923, it was recommended that a committee be appointed to revise the "Standard Methods for the Measurement of Concrete Work" (A. C. I. Proceedings, Vol. 9, 1913), or to formulate new rules to embody such changes in methods and construction as had taken place since that time. Committee C-5, as appointed, was made up of contractors, estimators, quantity surveyors, and those directly interested in a uniform method of measuring and estimating concrete work.

In preparing the new rules, the committee decided to use the 1913 standards as a basis, and to make such changes, revisions, and additions as were deemed necessary to conform to present day practice.

The committee work was carried on largely by correspondence, and the general procedure was to place a copy of the previous report with each member to be studied and returned with such changes and recommendations as were deemed necessary. These reports were then compared and corrected copies were mailed to members for further study. At the 1924 convention of the Institute, held in Chicago, the committee members present held several meetings to discuss these changes and make further revisions. A final committee meeting was held in New York, on Oct. 20, 1924, where final revisions were made, which were then submitted to all committee members and approved for final acceptance.

Concrete work was divided into five main classifications, and each of these was further sub-divided for estimating purposes:

1. Plain and reinforced structural concrete, such as is usually encountered in the construction of buildings, bridges and other engineering projects.

3. Structural precast concrete, including all concrete members which are cast on the ground and erected, rather than poured, in place.

2. Sidewalks and driveways, including such drives and walks as are placed directly upon the ground.

4. Cast ornamental concrete work, including all concrete work cast or completely manufactured in the shop, and sent to the job ready to be set in place.

5. Roads and pavements.

When considering and formulating the new rules the committee had to bear in mind at all times that the rules should be thoroughly practical if they were to be generally used, and classified in the same manner, and units as the work is actually constructed on the job or in the field.

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*The report was adopted as a Tentative Standard.

They should be definite on all points, inasmuch as they are to be used by the estimator and contractor in computing the quantities and costs of new work, and for this reason should produce estimates that are as nearly accurate as is humanly possible to prepare.

When used as a basis for unit price contracts, all chance of dispute should be eliminated by stating just what items shall make up each class of work, the method of measurement and the units generally used by the trade in present day practice.

When used as a basis of extras and credits between architect or engineer and contractor, they should form a fair basis for arriving at both extras and credits.

By estimating the work in practically the same units as the work is constructed on the job, it should enable the contractor to collect and compile costs in the same units in which the work is estimated. This would enable the contractor to compare estimated and actual costs during construction, which is important, if the contractor is to discover and rectify mistakes made in estimating.

It was also deemed advisable, insofar as practical, to estimate the various classes of work in the same units in which the materials are generally quoted and sold by the trade, viz., reinforcing steel by the ton, reinforcing mesh by the square foot, metal lath by the square yard, and concrete by the cubic yard rather than the cubic foot, inasmuch as practically all published tables state the number of barrels of cement, the fractional parts of cubic yards of sand and gravel required for one cubic yard of concrete, and as cement is almost universally sold by the barrel and sand and gravel by the cubic yard.

Problems of this kind, involving practically every material item and labor operation entering into the various classes of concrete work, were thoroughly discussed by the members attending the committee meetings, and inasmuch as they constituted the thought and general practice of many of the country's largest contractors as well as a liberal representation of the smaller ones, it is thought they express the views and opinions regarding the general practice of contractors in all parts of the country.

In presenting these new rules, the committee feels they should meet the requirements of the industry until such time as further changes in methods and construction would warrant further revisions or additions.

FRANK R. WALKER, *Chairman*
W. F. JENRICK, *Secretary.*

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specification on Measurement of and Estimating Concrete—Standard Methods for the Measurement of Concrete Work.

As submitted by Committee C-5.

Serial Designation C-5A-25 T.

The following divisions are recognized as separate and distinct operations in the construction of concrete work for which separate modes of measurement are necessary.

I. *Plain and Reinforced Structural Concrete:*

(a) Concrete
(b) Forms
(c) Reinforcement
(d) Surface Finish
(e) Tile Fillers

II. *Sidewalks and Driveways.*

III. *Structural Precast Concrete:*

(a) Concrete
(b) Reinforcement
(e) Erection

IV. *Cast Ornamental Concrete Work.*

V. *Roads and Pavements:*

The following general rules shall govern the measurement of the above items (with the exceptions where specifically noted):

(a) All work shall be measured net as fixed or placed in the structure.
(b) In no case shall non-existent material be measured to cover extra labor.
(c) No allowance shall be made for waste, voids, or cutting.

I. Plain and Reinforced Structural Concrete.

(a) Concrete.

1. The unit of measure for all concrete shall be the cubic yard.

2. All concrete shall be measured net as placed or poured in the structure, except as mentioned under Rule No. 4.

3. In no case shall an excess measurement of concrete be taken to cover the cost of forms or extra labor in placing.

4. All openings and voids in concrete shall be deducted with the following exceptions:

(a) No deduction shall be made for reinforcement, I-beams, bolts, etc., embedded in concrete except where a unit has a sectional area of more than 1 sq. ft.

(b) No deduction shall be made for pipes or holes in concrete having a sectional area of less than 1 sq. ft.

(c) No deduction shall be made for chamfered, beveled or splayed angles to columns, beams and other work, except where such chamfer, bevel or splay is more than 4 in. wide measured across the diagonal surface.

5. Each class of concrete having a different proportion of cement, sand or aggregate shall be measured and described separately.

6. Concrete in the different members of a structure shall be measured and described separately according to the accessibility, location or purpose of the work.

7. Concrete with large stones and rocks embedded in same (cyclopean masonry) shall be measured and described according to the richness of the mix and the percentage of rock in same.

8. Concrete stairs shall be measured by the linear foot of riser. Price to include forms, concrete, steel and finish tread. (Rise and tread to be given.) Finished surfaces shall be measured separately. In addition to stairs, measure platforms in square feet. Where stair risers are more than 5 ft. in length or stairs have unusual features, the concrete, forms and reinforcing should be measured and priced separately.

(b) Forms.

9. The unit of measure for formwork shall be the square foot of actual area of the surface of the concrete in contact with the forms or false work.

10. Forms to different parts of a structure shall be measured and described separately according to the position in the structure, accessibility, purpose and character of the work involved.

11. Forms shall in every case be measured and described separately and in no case (except as Rule 8) shall the measurement of concrete include the forms.

12. No deduction shall be made in measurement of surface of concrete supported by forms, because of forms being taken down and re-used two or three times in the course of construction.

13. The unit price for superficial measurement of forms shall include the cost of struts, posts, bracing, bolts, wire, ties, oiling, cleaning and repairing forms. No measurement to be made of these. Story heights over 12 ft. shall be listed separately, stating story height and area of forms.

14. Distinction shall be made between wood and metal forms.

15. Angle fillets or bevels to beams and columns shall be measured and described separately.

16. No deduction in measurement of forms shall be made for openings having an area of less than 25 sq. ft.

17. No deduction shall be made in floor forms for heads of columns of any shape, or area of clay or metal tile.

18. No deduction shall be made in column and girder forms for ends of girders, cross beams, etc.

19. No allowance shall be made for hand-holes in column forms for clearing out rubbish.

20. The measurement of column forms shall be the girth multiplied by the height from the floor surface to the under side of floor slab above or to the bottom of the drop panel.

21. Forms to rectangular, octagonal, hexagonal and circular columns shall be measured and described separately. Circular columns shall be listed separately, stating number, size and height of each.

22. Caps and bases to columns and other ornamental work shall be measured by number and fully described, giving overall dimensions.

23. The measurement of beam forms shall be the net length between columns multiplied by the sum of the breadth and twice the depth below the slab, except for beams at edge of floor or around openings which shall have the thickness of floor added to the sum of the breadth and twice the depth. The form area for the breadth of the beam should be deducted from the floor forms.

24. Allowance shall be made by number for pockets left for future beams.

25. Forms to circular work shall always be measured separately from forms to straight work.

26. No measurement or allowance shall be made for construction joints in slabs or beams, to stop the day's concreting.

27. Construction joints or expansion joints to dams, bridges and other large masses of concrete shall be measured by the square foot as they occur.

28. Falsework and staging for bridges, domes and other special work shall be described and measured separately.

29. Forms to cornices and moldings shall be measured by the lineal foot and the girth stated. (The term girth shall be taken to mean the total width of all curved and straight form surfaces touched by the concrete.) Plain forms to back of cornice to be measured separately.

30. Forms to window sills, copings and similar work shall be measured by the lineal foot. Indicate the dimensions.

31. If forms are required for the upper side of sloping slabs, such as saw-tooth roofs, they shall be measured separately.

(c) Reinforcement

32. The unit of measure of reinforcement shall be the weight in tons.

33. The weight shall be calculated on the basis of a square rod 1 in. x 1 in. x 12 in., weighing 3.4 lb.

34. Steel rods for reinforcement shall be measured as the gross weight required to be purchased.

35. Deformed bars shall be measured separately from plain.

36. Spirals shall be listed separately.

37. Separation shall be made according to accessibility, location or purpose of reinforcement.

38. The rods of each different size shall be measured and described separately.

39. Bent bars shall be measured separately from straight bars.

40. Chairs, ties, pipe sleeves, turnbuckles, clamps, threaded ends, nuts and other forms of mechanical bond shall be measured separately by number and size and allowed for in addition.

41. Wire cloth, expanded metal and other steel fabrics sold in sheets or rolls shall be measured and described by the square foot. The size of mesh and weight per square foot of steel in tension shall be stated. Allowance shall be made for waste, cutting, laps, etc., stating allowance.

(d) Surface Finish.

42. The unit of measure for finish or treatment of concrete surfaces shall be the square foot. The following shall be measured and described separately.

Cement wash. (State how many coats.)
Rubbing with carborundum.
Patching and removing of fins.
Scrubbing with wire brushes.
Tooling.
Picking.

Plastering.
Treating with acid.
Floor hardener.

43. Allowance shall be made for going over concrete work after removal of forms and patching up voids and stone pockets, and removing fins. Where rubbing is specified the cost of removing fins should be included with rubbing.

44. Cement or granolithic finish shall include all labor and materials for the thickness specified. Do not make deductions for partitions or columns. Do not deduct openings containing less than 25 sq. ft.

45. Finish laid integral with the slab shall be measured separately from finish laid after the slab has set.

46. Allowance shall be made for protection of finish with sawdust, sand or tenting, giving area to be covered.

47. Grooved surfaces, gutters, sills, curbing, etc., shall be measured separately from plain work and shall be measured by the square foot or lineal foot as the case may require.

48. Cement cove and base shall be measured by the lineal foot, giving size.

(e) Tile Fillers.

49. (a) Clay tile should be estimated by the number of square feet of each size tile required. Give number of end tile.

(b) Gypsum tile shall be measured by the number of lineal feet of each size. Soffit pieces shall be measured by the lineal foot, stating width.

(c) Metal tile shall be measured by the number of lineal feet of each size. State number of end tile pieces necessary.

(d) Wood tile shall be measured by the number of lineal feet of various depths.

(e) Metal lath or other ceilings under wood or metal tile ceilings shall be measured in square feet of surface covered.

II. SIDEWALKS AND DRIVEWAYS.

50. Sidewalks and driveways shall be measured by the square foot; the thickness to be stated in the description.

51. All linear surface measurement shall be made along the grade line or the actual length of the sidewalk or driveway and not merely horizontally.

52. Finish, reinforcing and lining in squares and cinder or stone foundations shall not be separately measured, but shall be described. Expansion joint material to be measured separately per linear foot.

53. Curbs and curb and gutter work shall be measured by the lineal foot and separated according to character and size, and shall include foundations, forms and finish.

54. In measuring curbs the full height and width or thickness of same shall be taken to the extreme edge.

55. Circular corners to curbs and gutters shall be measured separately by number, stating radius and length measured on the curve.

56. Vault lights shall be measured by the square foot, the size and type of glass, forms, steel and finish to be described but not measured separately. Beams under vault lights shall be measured by the lineal foot. In measuring vault lights the measurement shall be taken at least 4 in. beyond the outside line of the glass in each direction.

III. STRUCTURAL PRECAST CONCRETE.

(a) Concrete.

57. The term structural precast concrete is taken to include unit construction by the various systems.

58. The unit of measurement for structural precast concrete shall be the cubic foot, and shall be measured net as provided for monolithic concrete.

59. The various members shall be measured on the ground before erection.

60. No measurement shall be taken of forms.

(b) Reinforcement.

61. Reinforcement shall be measured separately as provided in Paragraphs 35 to 47, inc.

(c) Erection.

62. The unit of measure for the erection of structural precast concrete shall be the volume of the finished member in cubic feet.

63. The unit of measurement for grouting shall be the cubic foot of grout required.

IV. CAST ORNAMENTAL CONCRETE WORK.

64. Cast concrete shall be measured by the cubic foot, but the measurement shall be the smallest rectangular solid that will contain the piece measured and not its actual content.

65. No allowance shall be made for forms.

66. No allowance shall be made for reinforcement in trim and ornamental work.

67. No allowance shall be made for surface finish in trim and ornamental work.

68. Circular work shall be measured separately from other work.

69. Mitre blocks and end blocks for cornices, etc., shall be measured separately from straight molded work.

70. Vases, seats, pedestals, balusters and other similar items shall be measured by number and description with overall dimensions.

71. The unit of measure for the erection of ornamental concrete shall be the volume in cubic feet as measured under Rule 64.

V. Roads and Pavements.

72. The unit of measure for concrete pavement on roads or streets shall be the square yard of pavement surface.

73. All linear surface measurements shall be made along the grade line or the actual length of the pavement, and not merely horizontally.

74. Deductions shall be made for openings in concrete pavement of more than one square yard surface area. No deductions shall be made for individual openings, such as manholes, catch basins, inlets, lampholes, monument covers, etc., of less than one square yard surface area.

75. Concrete roads and pavements having sections of varying proportions of cement, sand and stone, or having sections of varying thickness or cross-section, shall be measured and described separately.

76. Concrete roads and pavements containing reinforcement of any type shall be measured and described separately from that not containing reinforcement.

77. Concrete roads and pavements requiring forms shall be measured and described separately.

78. Concrete roads or pavements requiring any special surface treatment shall be measured and described separately.

Report of Committee P-7, Concrete Pipe, Drain Tile and Conduit.

No meeting of the entire committee has been held since the 1924 convention of the Institute. However, three meetings of a sub-committee have been held for the purpose of revising the existing Tentative Standard Specifications for Reinforced-Concrete Sewer Pipe (P7C-24T), also to prepare other recommendations which are submitted below.

The report of the sub-committee has been sent out to all members of the committee and the result of the letter ballot is presented in the last paragraph of this report.

The committee recommends the following changes in: Tentative Standard Specifications for Concrete Drain Tile (P7B-24T), Tentative Standard Specifications for Plain Concrete Sewer Pipe (P7A-24T), and Tentative Standard Specifications for Reinforced-Concrete Sewer Pipe (P7C-24T).

Recommendation No. 1.*

That the Tentative Standard Specifications for Concrete Drain Tile (P7B-24T) be submitted to letter ballot of the Institute to become a standard of the Institute.

That the proposed Standard Specifications for Plain Concrete Drain Tile of 1910 and 1911 be withdrawn.

That the recommended practice for Plain Concrete Drain Tile of 1912 and the Standard Specifications for Concrete Drain Tile of 1917 be submitted to the Institute by letter ballot for withdrawal.

Recommendation No. 2.*

That the Tentative Standard Specifications for Plain Concrete Sewer Pipe (P7A-24T be continued as tentative standard for another year.

Recommendation No. 3.*

That the Tentative Standard Specifications for Reinforced-Concrete Sewer Pipe (P7C-24T) be amended as indicated on p. 580, where the revised specifications (P7C-25T) are found.

C. F. Buente, *Chairman*
M. W. Loving, *Secretary.*

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*Recommendations as suggested were adopted by the Institute Feb. 26, 1925.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications for Reinforced-Concrete Sewer Pipe.

Submitted by Committee P-7, Concrete Pipe.

(Serial Designation P-7C-25T)

I. General.

Scope and Classes.

1. These specifications apply to reinforced-concrete pipe intended to be used for the conveyance of sewage, industrial wastes and storm water. Pipe furnished under these specifications shall be of a single class as to strength.

Pipe may be made in either of two types (a) field-made cast concrete pipe or (b) shop-made mechanically-compacted pipe as hereinafter described.

Acceptability of Pipe.

2. The acceptability of pipe shall be determined by the purchaser; by the results of the load tests hereinafter specified, if and when required, and by inspection, to determine whether the pipe comply with the specifications as to dimensions, shape, and freedom from external and internal defects.

II. Materials.

Concrete and Steel.

3. Pipe shall be manufactured from concrete in which steel has been imbedded in such manner that the steel and concrete shall assist each other in taking stress.

By concrete is meant a suitable mixture of portland cement, mineral aggregates and water, hardened by hydraulic chemical action.

Cement.

4. Portland cement shall meet the requirements of the Standard Specifications and Tests for Portland Cement (Serial Designation, C 9-21) of the American Society for Testing Materials.

Steel

5. Reinforcement may consist of wire or fabric which will meet the requirements of the Tentative Specifications for Cold-Drawn Steel Wire for concrete reinforcement (Serial Designation: A 82-21 T) of the American Society for Testing Materials, or of rods and bars which meet the requirements of the Standard Specifications, for Billet Steel Concrete Reinforcement bars (Serial Designation A-15-15).

Fine Aggregate.

6. Fine aggregate shall consist of sand, stone screenings, or other inert materials with similar characteristics, or a combination thereof, having clean, hard, strong, durable uncoated grains and free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter, loam or other deleterious substances. Fine aggregate shall be well graded and shall pass a ¼-in. screen.

Coarse aggregate shall consist of crushed stone, gravel, slag, or other approved inert materials with similar characteristics or combinations thereof, having clean, hard, strong, durable, uncoated particles, free from injurious amounts of soft, friable, thin, elongated or laminated pieces, alkali, organic or other deleterious matter. **Coarse Aggregate.**

Where the shell thickness is 5 inches or less, coarse aggregate shall be used which will pass a screen having one-inch openings. On 5½ to 9-in. shells, the stone may pass a screen having openings of 1½ in.

7. The mix shall be such as to produce a pipe that will meet the physical test requirements hereinafter provided, but under no circumstances shall the mixture consist of less than 1 part portland cement to 2 parts fine and 4 parts coarse aggregate with a minimum amount of water to produce a workable mix. **Mixture.**

TABLE I.—MINIMUM DIMENSIONS OF FIELD-CAST REINFORCED-CONCRETE PIPE.

Interior Diameter in Inches	Shell Thickness in Inches	Total Min. Cross-Sect. Area of Steel sq. in. per lin. ft. of pipe	
24	3	1 Line	.065
27	3	"	.065
30	3½	"	.085
33	3¾	"	.105
36	4	"	.125
42	4½	"	.150
48	5	2 Lines	.210
54	5½	Totaling	.250
60	6	"	.290
66	6½	"	.320
72	7	"	.360

III. DESIGN.

8. Field-made cast concrete pipe shall be made with concrete having a consistency which will permit the concrete to be placed in the forms and will require spading to properly work it around the reinforcement. The pipe may be made with wall thicknesses and steel quantities shown in Table I provided they meet all the strength and other requirements of these specifications. It is clearly understood that the wall thicknesses and quantities of steel given in this table are not specified wall thicknesses and steel quantities, but are minimums and that manufacturers shall use greater shell thicknesses or greater steel quantities or both if necessary to meet all other requirements. **Field Made Cast Concrete Pipe.**

9. Shop-made mechanically-compacted pipe must be manufactured in a factory and by a process which will automatically compact the concrete into the forms and around the reinforcement in such a manner as to uniformly obtain the maximum quality and strength for the lesser amount of concrete allowed. **Shop-Made Mechanically-Compacted Pipe.**

The pipe may be made with wall thicknesses and steel quantities shown in Table II provided they meet all the strength and other requirements of these specifications. It is clearly understood that the wall thicknesses and quantities of steel given in this table are not specified wall thicknesses and steel quantities, but are minimums and the manufacturers shall use greater shell thicknesses or greater steel quantities or both if necessary to meet all other requirements.

It is clearly understood that these requirements constitute factory conditions and, therefore, pipe shall not be considered as meeting these specifications unless made at a plant established for permanent operation.

TABLE II.—MINIMUM DIMENSIONS OF SHOP-MADE MECHANICALLY-COMPACTED REINFORCED-CONCRETE PIPE.

Interior Diameter in Inches	Shell Thickness in Inches	Total Min. Cross-Sect. Area of Steel sq. in. per lin. ft. of pipe	
24	2½	1 Line	0.15
27	2⅝	"	0.18
30	2¾	"	0.21
33	2¾	"	0.24
36	3	2 Lines	0.36
42	3⅜	Totaling	0.42
48	3¾	"	0.46
54	4⅛	"	0.50
60	4½	"	0.54
66	4¾	"	0.58
72	5	"	0.62

Reinforcement. 10. The reinforcement shall extend throughout the barrel of the pipe. It shall be assembled into units so designed that they may be readily placed and maintained of true, exact shape and proper position within the pipe form during the manufacturing process.

Joints. 11. The ends of reinforced concrete pipe shall be so formed that when the pipe are laid together and the joints cemented, they shall make a continuous and uniform line of pipe with a smooth and regular interior surface. The joints shall be of such a design that when cemented they will prevent leakage and infiltration as well as appreciable irregularities in the flow line of the sewer.

Position of Reinforcement. 12. Reinforcement shall be placed not less than ¾ in. nor more than 1¼ in. from the inner surface of the pipe shell when one line of steel is used and from either surface when two lines are used.

The reinforcement may be of elliptical form and placed in pipe near the inner surface on the vertical axis and near the outer surface on the horizontal axis of the pipe, providing all other requirements given in Tables I and II are complied with.

IV. WORKMANSHIP AND FINISH.

13. Pipe shall be substantially free from fractures, large or deep cracks and surface roughness. The ends of pipe shall be square with their longitudinal axes. **Finish.**

14. Variations of the internal diameter shall not exceed 1.5 per cent. The shell thickness shall not be less than that intended in the design by more than 5 per cent. at any point. **Variations.**

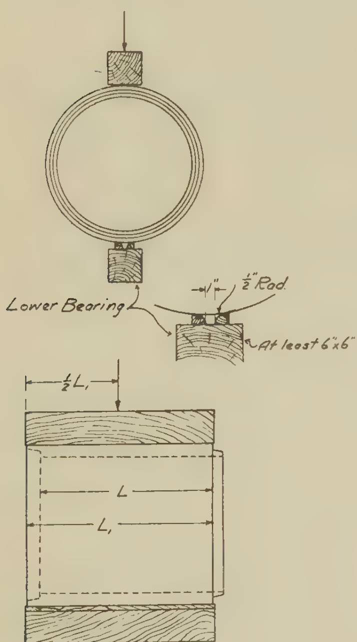

FIG. 1.—THREE-EDGE BEARINGS.

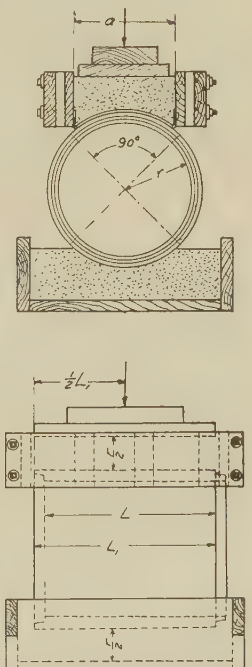

FIG. 2.—SAND BEARINGS.

V. MARKING.

15. When shipment of pipe is made in any manner other than direct from manufacturer to user, pipe shall be so marked that the manufacturer of the pipe can be identified. The date of manufacture shall be plainly marked on the pipe in all cases. **Markings.**

VI. LOAD TEST.

16. The test specimens shall be full size pipe which will in every respect pass the inspection requirements hereinafter provided. **Quality of Specimen.**

Selection of Specimen. 17. The specimens to be tested shall be selected by the purchaser or his representative at the point or points designated by him when placing the order. The manufacturer shall furnish, for testing purposes and at his own expense, one pipe of each size included in the order; the purchaser bearing all expense of testing such pipe. Should additional tests be made upon the demand of the purchaser or manufacturer, as hereinafter provided, then cost of such additional test specimens and the expense of testing shall be borne by the party making such demand.

Complying with Test. 18. Should the test specimens furnished by the manufacturer meet the test requirements, then all pipe represented by such specimens shall be

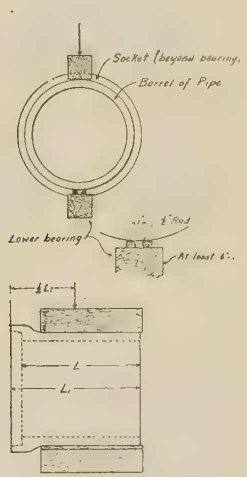

FIG. 3.—THREE-EDGE BEARINGS.

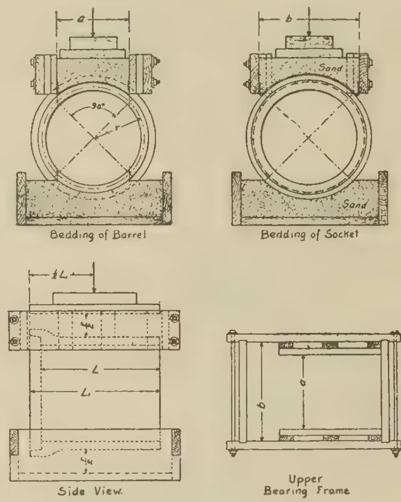

FIG. 4.—SAND BEARINGS.

accepted; provided, however, that the purchaser shall have the right to require an additional test in any size or sizes of pipe.

Should any of the specimens first tested fail to meet the test requirements, then the manufacturer shall have the right to submit an additional test of the size or sizes of pipe which have failed.

In either case, when an additional test is made in any size of pipe, such test shall consist of three pipe or multiple thereof including the pipe first tested; if two-thirds of the pipe so tested shall meet the test requirements, then all pipe represented by such test shall be accepted, otherwise they shall be rejected.

Test Methods 19. The load test shall be performed either by the sand bearing or three-edge bearing methods as hereinafter described.

Sand Bearing Test. 20. When sand bearings are used, the specimen shall have its upper and lower quadrants, measured on the center line of the shell, carefully

bedded in dry, clean, uniformly compacted sand having a minimum thickness of at least one-quarter the mean diameter of the pipe to be tested. The sand shall be prevented from lateral flow by being placed in rigid frames which shall not come in contact with the test specimen. Seepage of sand between the pipe and the upper bearing frame may be prevented by strips of cloth attached to the lower inside edge of the frame.

The upper bearing shall be centered over the pipe and the load shall be applied centrally thereto through a rigid horizontal bearing plate, covering its surface but not in contact with the bearing frame.

Test by Weights.

21. The sand bearing test may be made without the use of a testing machine, by piling weights directly on a platform resting on, and fully supported by, the top bearing plate, provided, however, that the weights shall be placed symmetrically about a vertical line through the center of the pipe, and that the platform shall not be allowed to touch the top bearing frame.

Three-edge Bearing Test.

22. When three-edge bearings are used, the ends of each specimen of pipe shall be accurately marked in halves of the circumference prior to the test. The lower bearings shall consist of two wooden strips with vertical sides, each strip having its interior top corner rounded to a radius of approximately ½ in. They shall be straight, and shall be securely fastened to a rigid block with their interior vertical sides one inch per foot of diameter of pipe apart. The upper bearing shall be a wooden block, straight and true from end to end. The bearings shall be centered on the diametrically opposite markings previously made and the test load shall be applied through the upper bearing block in such a way as to produce a uniform distribution at the load throughout the length of the pipe and to leave the bearing free to move in a vertical plane passing midway between the lower bearings.

Application of Load (Machine Testing)

23. Any prime mover or hand power which will apply the load at a uniform rate of about 2000 lb. per minute, or in increments of not more than 100 lb., at the same rate, may be used in making the test. The testing machine shall be substantial and rigid throughout, so that the distribution of the load will not be affected appreciably by the deformation or yielding of any part. The load shall be applied continuously until the ultimate strength of the pipe is reached. The load at ultimate strength and also when the first crack appears shall be observed and recorded.

24. In testing pipe which is "out of line" the lines of the bearings chosen shall be from those which appear to give the most favorable conditions for fair test.

Test Result.

25. The test specimens shall show an ultimate strength not less than that given in Table III for the various sizes and methods of test stated and shall show no clearly visible crack, caused by the application of the load, extending the full length of pipe when tested to one-half the ultimate load designated in Table III. Ultimate load is the point beyond which no additional load can be sustained.

TABLE III.—ULTIMATE STRENGTH TEST REQUIREMENTS OF REINFORCED-CONCRETE SEWER PIPE AT NOT LESS THAN 28 DAYS.

Internal Diameter of Pipe in Inches	Ultimate Load in Pounds per lin. ft. of Pipe	
	Three-edge Bearing	Sand Bearing
24	3000	4500
27	3300	4950
30	3600	5400
33	3900	5850
36	4200	6300
42	4700	7050
48	5100	7650
54	5500	8250
60	5800	8700
66	6000	9000
72	6200	9300

NOTE: The load per foot of pipe shall be determined by dividing the total test load by the net inside length of the barrel of the pipe, measuring from the bottom of the socket to the end of the spigot.

VII. INSPECTION.

26. All pipe shall be subject to inspection at the factory, trench or other point of delivery by a competent inspector employed by the consumer or purchaser. The purpose of the inspection shall be to cull and reject pipe which, independent of the physical tests herein specified, fail to meet the requirements of these specifications.

27. Pipe shall be subject to rejection on account of any of the following:

(a) Variations in any dimension exceeding the permissible variations given in Section 13.

(b) Fractures or cracks passing through the shell, except that cracks at either end of the pipe not extending beyond the joint nor fracture not extending beyond the joint nor extending more than 10 per cent around the circumference, will not be considered cause for rejection.

(c) Defects which indicate imperfect mixing and molding.

(d) Exposure of the reinforcement when such exposure would indicate that the reinforcement is misplaced.

28. All rejected pipe shall be plainly marked by the inspector and shall be replaced by the manufacturer or seller with pipe which meets the requirements of these specifications, without additional cost to the consumer or purchaser.

AMERICAN CONCRETE INSTITUTE STANDARD.

Standard Specifications for Portland Cement Concrete Sidewalks.*

(Serial Designation *C-2B-25*)

Submitted by Committee C-2 on Concrete Floors.

I. Origin and Use.

(A) Origin.

1. These specifications as printed herein were revised and arranged by Committee C-2 on Concrete Floor Finish. The Tentative Standard Specifications for Portland Cement Concrete Sidewalks were printed in *Proceedings* of the American Concrete Institute, Vol. 20, 1924, p. 732. **Origin.**

(B) Use of Specifications.

2. These specifications cover both one- and two-course sidewalk construction. The specifications for materials and construction included in Paragraphs 1 to 36 inclusive, apply to both one- and two-course types of sidewalks. The use of the specifications will be facilitated by the following subdivision of them by paragraphs according to the type of sidewalk involved: **Use.**

For one-course sidewalks use Paragraphs 3 to 41, inc.

For two-course sidewalks use Paragraphs 3 to 38, inc., and 42 to 45, inc.

II. Materials.

3. Before delivery upon the job, and at such other times as the engineer deems necessary, the contractor shall furnish any required samples of the materials hereafter mentioned. Materials shall pass the following requirements. **General.**

(A) Cement.

4. The cement shall meet the requirements of the current Standard Specifications for Portland Cement of the American Society for Testing Materials. **Cement.**

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*This specification has been a tentative standard for one year. The convention, on Feb. 26, adopted it to be sent to letter ballot as standard. Letter ballot was canvassed May 28, 1925.

(B) Fine Aggregate.

Fine Aggregate

5. The fine aggregate shall consist of clean, hard, durable, uncoated particles of sand, stone or air-cooled blast furnace slag, free from organic material. All of the fine aggregate shall pass a ¼-in. screen and 95 per cent shall be retained on a 100-mesh screen. Not more than 25 per cent shall pass a 50-mesh screen. It shall be well graded from coarse to fine, and shall contain not more than 5 per cent by weight of clay or loam, none of which shall be in lumps. Blast furnace slag used in the wearing course shall weigh not less than 70 lb. per cubic foot.

(C) Coarse Aggregate.

General.

6. Coarse aggregate may be broken stone, gravel or blast-furnace slag. All of the coarse aggregate shall pass a 1-in. screen and at least 95 per cent shall be retained on a ¼-in. screen, with all intermediate sizes retained.

Broken Stone or Gravel.

7. The broken stone or gravel shall be clean, hard, durable, uncoated rock. It shall contain no vegetable or other deleterious matter and shall be practically free from soft, thin, elongated or laminated pieces.

Slag.

8. Slag shall be air-cooled blast-furnace slag weighing not less than 65 lb. per cubic foot when used in that part of the sidewalk not subject to abrasion, and not less than 70 lb. per cubic foot when used in the wearing course.

(D) Water.

9. Water shall be clean and practically free from alkali, oils or acid.

(E) Joint Filler.

Joint Filler.

10. The joint filler shall be a suitable elastic waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather, or prepared strips of fiber matrix and bitumen as approved by the engineer. The strips shall be ½ in. in thickness, their width shall at least equal the full thickness of the slab and their length shall at least equal the width of the slab at the joint.

(F) Forms.

General.

11. Forms shall be of lumber 2 in. thick, or of steel of equal strength, except on curves, where flexible strips may be used.

(G) Division Plates.

Division Plates.

12. Where division plates are used they shall be of ⅛-in. steel as wide as the depth of the slab and as long as the width of the walk.

(H) Sub-base.

Sub-base.

13. Only clean, durable material, such as coarse gravel or steam-boiler cinders free from ash or particles of unburned coal shall be used in the sub-base. (Note: Eliminate this clause if sub-base is not required.)

III. CONSTRUCTION.

(A) Subgrade.

14. That portion of the ground surface directly beneath the slab shall be called the subgrade. **Definition.**

15. All soft and spongy material in the subgrade shall be removed and replaced with suitable material. Fills shall be compacted in layers not exceeding 6 in. in thickness. Spots previously compacted by traffic shall be loosened to a depth of 6 in. The whole subgrade shall be thoroughly and uniformly compacted to a firm surface having as nearly as possible a uniform bearing power. **Preparation of Subgrade.**

16. A templet, resting upon the side forms and having its lower edge at the exact elevation of the subgrade, shall be drawn along the forms before any concrete is laid. Any high places in the subgrade shall be removed so that the templet will pass over without being raised off the side forms or being tipped at an angle to the sidewalk surface. **Surface of Subgrade.**

17. The subgrade shall be damp, but not muddy, when concrete is placed upon it. **Wetting.**

(B) Drains.

18. Where the nature of the subgrade is such as to produce unusual frost action, drains of 4-in. concrete tile shall be laid on the lines and grades given by the engineer.

(C) Sub-base.

19. The sub-base material shall be spread, thoroughly rolled and tamped to a surface at least 5 in. below the finished grade of the walk. On fills, the sub-base shall have the same direction of slope as the surface of the fill. **Applying Material**

20. While compacting the sub-base, the material shall be kept thoroughly wet, and shall be wet when the concrete is deposited, but shall not show pools of water. **Wetting.**

(D) Forms.

21. Forms shall be held rigidly in place by stakes or braces with top edges at true line and grade, to give the walk a slope toward the curb of ¼ in. per foot of width. Ends of adjoining forms shall be flush. **Forms.**

22. Forms and division plates shall be thoroughly cleaned and oiled each time before they are used. **Cleaning and Oiling.**

(E) Measuring and Mixing.

23. The method of measuring the materials for the concrete or mortar, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 lb. net) shall be considered one cubic foot. Cement shipped in bulk shall be proportioned by weight. **Measuring.**

Machine Mixing. 24. All concrete shall be mixed by machine except when the architect or engineer shall otherwise permit under special conditions. A batch mixer of an approved type shall be used. The ingredients of the concrete or mortar shall be mixed to the specified consistency, and the mixing shall continue for at least one minute after all materials are in the drum. The drum shall be completely emptied before receiving material for the succeeding batch.

Hand Mixing. 25. When it is necessary to mix by hand, the materials shall be mixed dry on a watertight platform until the mixture is of uniform color, the required amount of water added, and the mixing continued until the mass is of uniform consistency and homogeneous.

Retempering. 26. Retempering of mortar or concrete which has partially hardened, that is, remixing with or without additional materials or water, shall not be permitted.

Consistency. 27. The consistency of the mixed concrete shall be such that no separation of the ingredients takes place and some tamping is necessary to bring the water to the surface.

(F) Cold Weather Work.

Cold Weather Work. 28. Concrete shall not be placed on a frozen subgrade or when the temperature is, or is liable to be within 24 hours, below 35 deg. F., except with the written permission of the engineer, and according to his instructions. Freshly-placed concrete shall not be allowed to freeze for a period of five days after placing.

(G) Jointing.

Construction Joints. 29. The walk shall be cut into separate rectangular slabs. No plain concrete slab shall be longer than 6 ft. on any one side.

Division Plates. 30. Division plates shall be removed after the concrete has hardened sufficiently to avoid breaking the edges or corners of the slabs.

Cut Joints. 31. Where division plates have not been used, the partially set concrete shall be cut through to the subgrade or sub-base. Care shall be taken to make the cut at right angles to the surface of the sidewalk.

Edges. 32. The surface edges of each slab shall be rounded to a radius of about ¼ in. Markings shall be exactly at cuts between slabs.

(H) Expansion Joints.

General. 33. Expansion joints shall extend from the surface to the subgrade, be truly at right angles to the sidewalk surface and be made by putting the specified joint filler in place before placing the concrete. They shall be placed as follows:

At Curb Line. 34. At or near all places where sidewalk line intersects a curb line or other sidewalk a 1-in. expansion joint shall be made at right angles to the center line of the walk.

35. When the sidewalk fills the space between the curb and the building line a ½-in. expansion joint shall be placed between the curb and the sidewalk, and between the sidewalk and the building. **Longitudinal Expansion Joints.**

36. A ½-in. expansion joint shall be made across the walk at approximately 50-ft. intervals. **Cross Joints.**

(1) Curing and Protecting.

37. As soon as the concrete has set sufficiently it shall be sprinkled and kept moist until covered. As soon as it can be done without damage to the walk it shall be covered with 2 in. of earth or sand which shall be kept wet seven days. The walk shall then be cleaned and opened to traffic.

38. The contractor shall protect the concrete from damage by rain, pedestrians and animals, with suitable covers and barricades, and by red lights at night.

IV. ONE-COURSE WORK.

39. The sidewalk shall consist of one 5-in. course of concrete in the proportion of one part of portland cement, two parts of fine aggregate and three parts of coarse aggregate. **Thickness and Proportions.**

40. The freshly-mixed concrete shall be placed promptly on the prepared subgrade. It shall then be struck off and tamped with a straightedge resting upon the side forms and advanced with a crosswise sawing motion. Concrete may be delivered in trucks from a centrally located mixing plant provided the consistency of the concrete and the length of haul are such that the concrete can be delivered in a homogeneous and workable condition. **Placing.**

41. The concrete shall then be floated with a wooden float until the surface is true and the concrete thoroughly compacted. **Finishing.**

V. TWO-COURSE CONSTRUCTION.

42. Two-course sidewalks shall consist of a base 4¼ in. thick composed of concrete in the proportions one part portland cement, three parts of fine aggregate and five parts coarse aggregate and a top coat ¾ in. thick composed of mortar in the proportions one part portland cement and two parts fine aggregate. **Thickness and Proportions.**

43. The base shall be deposited on the subgrade and thoroughly compacted by tamping. It shall then be struck off by a templet which shall leave it nowhere less than ¾ in. below the finished surface. **Laying Base.**

44. Within 45 minutes after the bottom course is laid and before the initial set has taken place, the material for the wearing surface shall be placed and brought to the established grade by means of a strike board. **Placing Wearing Surface.**

45. After the wearing course has been brought to the established grade it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. **Finishing.**

PROPOSED REVISION STANDARD SPECIFICATIONS FOR CONCRETE FLOORS.*

(SERIAL DESIGNATION *C-2A-23*)

D. COARSE AGGREGATE.

Coarse Aggregate.

7. Coarse aggregate shall consist of clean, hard, tough, crushed rock, pebbles or air-cooled blast furnace slag, graded in size, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall range from one and one-half (1½) inches down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed. The weight per cubic foot of blast-furnace slag shall not be less than 65 lb. per cubic foot in that portion of the slab not subject to abrasion, and not less than 70 lb. per cubic foot for use in the wearing surface.

A. C. IRWIN, *Secretary.*

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*Committee C-2 on Concrete Floors submitted the following recommended change in the Standard Specifications for Concrete Floors (C2A-23). The proposed revision was accepted by the convention Feb. 26 and Paragraph D-7 therefore becomes a Tentative Standard.

Report of Committee P-1, On Standard Building Units.

Your committee has invited comment and constructive criticism relating to the specifications and forms on which the committee has been working. Careful consideration has been given to all communications addressed to the chairman or secretary of the committee and a careful study has been made of the specifications submitted to the Institute at the convention in 1924. It was the intent of the committee to have concrete building block, concrete building tile and concrete brick tested for compressive strength in a dry state. While we believe that this intent will be apparent to any one who carefully studies the specifications, yet we feel that it will be wise to change the wording so as to clarify this thought and also simplify the wording of the specifications.

Building Block and Tile.—Paragraph 11 of the Proposed Standard Specifications for Concrete Building Block and Concrete Building Tile now reads:

"The specimens used in the absorption test may be used for the strength test, provided they have been dried at approximately 70 deg. F. for not less than three days.

"We recommend that Paragraph 11 shall read:

"The specimens used in the absorption test may be used for the strength test," thereby deleting the words beginning with "provided" and ending with "three days."

We recommend that Sec. 2 be rearranged as to position of paragraphs and that the paragraph shall read as shown in the Specifications on P. 603.

We submit herewith copy of the Proposed Standard Specifications for Concrete Building Block and Concrete Building Tile, as they would be after the proposed changes are made. We recommend that the amended specifications be adopted as standard, inasmuch as these changes do not in effect change the intent of the present specifications.

We submit herewith a revision of the Proposed Standard Specifications for Concrete Brick, the revision being made simply to clarify the intent of the specifications relative to testing for compressive strength in a dry condition, the same as was done for the specifications for Concrete Building Block and Concrete Building Tile. As the intent of the specifications has not been changed in any particular, we, therefore, recommend that the Proposed Standard Specifications for Concrete Brick as revised be now adopted as standard, subject to letter ballot.

Concrete Brick.—We submit herewith a revision of the Proposed Standard Specifications for Concrete Brick, the revision being made simply to clarify the intent of the specifications relative to testing for compressive

strength in a dry condition, the same as was done for the specifications for Concrete Building Block and Concrete Building Tile. As the intent of the specifications has not been changed in any particular, we, therefore, recommend that the Proposed Standard Specifications for Concrete Brick as revised be now adopted as standard, subject to letter ballot.

Standardized Sizes.—At the last convention this committee suggested that it would be of benefit to the concrete products industry and to the users of the products, if standard sizes were adopted by the Institute and by the manufacturers of the products.

Since the last convention this committee, through a subcommittee, has co-operated with the Division of Simplified Practice of the United States Department of Commerce. At the 1924 convention this committee recommended that sizes of block, tile and brick should be as follows:

Standard Sizes for Concrete Block:

Height	Width	Length
7⅝ in.	6 in.	15⅝ in.
7⅝ "	8 "	15⅝ "
7⅝ "	10 "	15⅝ "
7⅝ "	12 "	15⅝ "

Standard Sizes for Concrete Building Tile:

Height	Width	Length
5 in.	3¾ in.	12 in.
5 "	8 "	12 "
5 "	12 "	12 "

Standard Sizes for Concrete Brick:

	ROUGH	
Height	Width	Length
2¼ in.	3¾ in.	8 in.
	SMOOTH	
2¼ in.	3⅞ in.	8 in.

These recommended sizes were transmitted to the Division of Simplified Practice for their approval, and the division called a general conference October 16 at the Hotel Sherman, Chicago. Approximately eighty invitations were issued to all interests, including concrete products and machinery manufacturers and the findings of Committee P-1 were used as the basis of discussion.

After a great deal of deliberation, it was the consensus that certain sizes of concrete block be adopted with an allowance of sufficient tolerance to permit either the use of ¼ or ⅜-in. mortar joint. The adoption of this tolerance changed the recommended sizes of these blocks as submitted by Committee P-1.

The subject of concrete building tile was next considered and the recommendation of Committee P-1 was unanimously adopted. As these sizes only pertain to load bearing wall tile, it was the vote of the conference that the same sizes of standard partition tile as agreed upon by the clay tile industry would also meet the entire demand for concrete tile. Upon vote, duly seconded, six sizes of partition tile were adopted by the conference.

Discussion on concrete brick developed the fact that there was no necessity for two sizes. A motion, duly seconded and unanimously passed, eliminated the 3⅞-in. width as recommended by Committee P-1. As a result of this action, only one size of concrete brick, both face and common, was adopted as standard.

The conference went on record that these recommendations should become effective June 1, 1925, continuing in effect for the period of one year.

It further requested the Division of Simplified Practice to circularize the industry for its acceptance and to appoint a standing committee of the conference. As soon as 80 per cent acceptance of those circularized is received by the Department, these recommendations will be published in its "Elimination of Waste" series.

The U. S. Department of Commerce, through the Bureau of Standards, recommends that the number of sizes of concrete building units be reduced to the following:

TABLE 1—CONCRETE BLOCK.

Height	Tolerance	Width	Tolerance	Length	Tolerance
7¾ in.	Minus ⅛ in.	6 in.	Minus ¼ in.	15¾ in.	Minus ⅛ in.
7¾ "	Minus ⅛ "	8 "	Minus ¼ "	15¾ "	Minus ⅛ "
7¾ "	Minus ⅛ "	10 "	Minus ¼ "	15¾ "	Minus ⅛ "
7¾ "	Minus ⅛ "	12 "	Minus ¼ "	15¾ "	Minus ⅛ "

TABLE 2—CONCRETE BUILDING TILE.

	Height	Width	Length
Load Bearing	5 in.	3¾ in.	12 in.
	5 "	8 "	12 "
	5 "	12 "	12 "
Partition	3 "	12 "	12 "
	4 "	12 "	12 "
	6 "	12 "	12 "
	8 "	12 "	12 "
	10 "	12 "	12 "
	12 "	12 "	12 "

Note.—Not more than 3 per cent permissible variation over or under for dimensions covering height, width or length.

TABLE 3—CONCRETE BRICK.

Types	Height	Width	Length
Face and Common	2¼ in.	3¾ in.	8 in.

(Name of Laboratory)

TESTS OF CONCRETE BUILDING UNITS

Manufacturer:..

Client:..

Type of Unit:..

Tests made in accordance with Tentative Standard Specifications for Concrete Building Block and Concrete Building Tile, and Tentative Standard Specifications for Concrete Brick, of the American Concrete Institute, 1924. See reverse of this sheet.

ABSORPTION TESTS

Mark		Date Tested	Dry Weight of Unit pounds	Absorption percent by wt.	Unit Weight of Concrete lb. per cu. ft.*	Corrected Absorption percent*
Lab.	Client					
Average						

* This information need not be furnished if the absorption by weight is within the required limits.

STRENGTH TESTS

Mark		Date Tested	Age at Test Days	Dimensions of Unit in.			Loaded Area, sq. in.		Crushing Load		
									Total Pounds	Lb. per sq. in.	
Lab.	Client			Depth	Width	Length	Gross	Minimum		Gross Area	Minimum Area
Average											

Remarks:

Correct........................

Approved......................

Date of Report.................... Report No...........

Detach here if following information is confidential.

Report No...........

INFORMATION FURNISHED BY CLIENT

DATA OF CONCRETE

Mark	Date Made	Types of Machinery		Time of Mixing	Mixing Water per Sack of Cement	Units per Sack of Cement	Proportions of Cement to Fine and Coarse Aggregate	Time and Method of Curing
		Mixer	Molding					

DATA OF AGGREGATES

Mark	Type of Aggregate	Sieve Anlayses Amounts Coarser than each Sieve, Per cent by Weight								Fineness Modulus	Unit Weight, lb. per cu. ft.		Moisture in Aggregate as Used, percent by weight
		100	50	30	16	8	4	⅜	¾		Dry and Rodded	As Used	

REMARKS

(Over)

FIG. 5.—ONE SIDE OF FORM FOR REPORTING TESTS ON CONCRETE BUILDING UNITS.

(The other side has copy of Standard Specifications.)

It will be seen by a study of the Table 1, that if we assume that the minus tolerance permits of two sizes of concrete block, then the adoption of the standard sizes for concrete block would reduce the lengths from 30 in number as now found in practice, to 2 in number, the number of widths from 20 to 8 and the number of heights from 26 to 2. Variation in each size is but ⅛ in. in any dimension whether length, width or height.

Referring to Table 2, it will be noted that reduction in tile sizes would be made from eight lengths to one length and from seven widths to three widths and from five heights to one height for load-bearing tile, whereas, in partition tile there are but six heights, one width and one length.

Referring to Table 3, concrete brick, it will be found that the standardization of sizes would cause a reduction from six lengths to one length, from seven widths to one width and from six heights to one height.

We recommend that the members here assembled discuss these sizes if they are so inclined and we recommend the adoption of the standard sizes set forth in this report in Tables 1, 2 and 3.

Forms for Reporting Tests on Concrete Building Units.—A subcommittee, consisting of Stanton Walker, has made a study of the form submitted at the last convention, and this committee recommends the changes shown in the attached copy. We also recommend that the word "tentative" shown on the attached copy be deleted if the proposed specifications for building units be adopted, otherwise the word tentative" should stand. We therefore recommend the adoption of the form of report as shown in Fig. 1.

This committee has been studying the preparation of specimens for testing and after making some tests recommends that capping as provided in the proposed standard specifications is satisfactory and that no change should be made.

AMERICAN CONCRETE INSTITUTE STANDARD.

Standard Specifications for Concrete Building Block and Concrete Building Tile.*

Submitted by Committee P-1, on Standard Building Units.

(Serial Designation *P-1A-25*)

1. GENERAL.

1. The purpose of these specifications is to define the requirements for concrete building block and concrete building tile to be used in construction.

2. The word "concrete" shall be understood to mean portland-cement concrete.

Strength Requirements.

3. According to the strength in compression 28 days after being manufactured or when shipped, concrete block and concrete tile shall be classified as heavy load bearing, load bearing, and non-load bearing on the basis of the following requirements:

	Compressive strength, lbs. per sq. in. of gross cross-sectional area as laid in the wall.	
	Aver. of 3 or more units.	Min. for Individual Unit.
Name of Classification		
Heavy load bearing block or tile.......	1200	1000
Medium " " " " ".......	700	600
Non- " " " " ".......	250	200

4. The gross cross-sectional area of a one-piece concrete block or tile shall be considered as the product of the length times the width of the unit as laid in the wall. No allowance shall be made for air spaces in hollow units. The gross cross-sectional area of each unit of a two-piece block or tile shall be considered the product of the length of the unit times one-half the thickness of the wall for which the two-piece block or tile is intended.

5. The compressive strength of the concrete in units of all classifications except "non-load bearing block" shall be at least 1000 lb. per sq. in., when calculated on the minimum cross-sectional area in bearing.

Absorption Requirements.

6. Concrete Building Block and tile to be exposed to soil or weather in the finished work (without stucco, plaster or other suitable protective covering) shall meet the requirements of the absorption test.

7. All concrete building block and tile not covered by paragraph 6 need not meet an absorption requirement.

8. Concrete block and tile shall not absorb more than 10 per cent of the dry weight of the unit when tested as hereinafter specified, except when it is made of concrete weighing less than 140 lb. per cu. ft. For

*This specification has been a Tentative Standard for a year. The convention, on Feb. 26, 1925, adopted it to be sent to letter ballot as standard. Letter ballot was canvassed May 28, 1925.

block or tile made with concrete weighing less than 140 lb. per cu. ft. the absorption in per cent by weight shall not be more than 10 multiplied by 140 and divided by the unit weight in pounds per cubic foot of the concrete under consideration.

Sampling.—9. Specimens for tests shall be representative of the commercial product of the plant.

10. Not less than three and preferably five specimens shall be required for each test.

11. The specimens used in the absorption test may be used for the strength test.

2. METHODS OF TESTING.

Absorption Test.

Absorption Test.—12. The specimens shall be immersed in clean water at approximately 70 deg. F. for a period of 24 hours. They shall then be removed, the surface water wiped off, and the specimens weighed. Specimens shall be dried to a constant weight at a temperature of from 212 deg. to 250 deg. F. and reweighed. Absorption is the difference in weight divided by the weight of the dry specimens and multiplied by 100.

Weight of Concrete.—13. The weight per cubic foot of the concrete in a block or tile is the weight of the unit in pounds divided by its volume in cubic feet. To obtain the volume of the unit fill a vessel with enough water to immerse the specimen. The greatest accuracy will be obtained with the smallest vessel in which the specimen can be immersed with its length vertical. Mark the level of the water, then immerse the saturated specimen and weigh the vessel. Draw the water down to its original level and weigh the vessel again. The difference between the two weights divided by 62.5 equals the volume of the specimen in cubic feet.

Strength Test.

Strength Test.—14. Specimens for the strength test shall be dried to constant weight at a temperature of from 212 deg. to 250 deg. F.

15. The specimens to be tested shall be carefully measured for overall dimensions of length, width and height.

16. Bearing surfaces shall be made plane by capping with plaster of paris or a mixture of portland cement and plaster which shall be allowed to thoroughly harden before the test.

17. Specimens shall be accurately centered in the testing machine.

18. The load shall be applied through a spherical bearing block placed on top of the specimen.

19. When testing other than rectangular block or tile care must be taken to see that the load is applied through the center of gravity of the specimen.

20. Metal plates of sufficient thickness to prevent appreciable bending shall be placed between the spherical bearing block and the specimen.

21. The specimen shall be loaded to failure.

22. The compressive strength in pounds per square inch of gross cross-sectional area is the total applied load in pounds divided by the gross cross-sectional area in square inches.

AMERICAN CONCRETE INSTITUTE STANDARD.

Standard Specifications for Concrete Brick.*

Submitted by Committee P-1, on Standard Building Units.

(Serial Designation *P-1B-25*)

1. GENERAL.

1. The purpose of these specifications is to define the requirements for concrete brick to be used in construction.

2. The word "concrete" shall be understood to mean portland-cement concrete.

3. The average compressive strength of concrete brick 28 days after being manufactured or when shipped shall not be less than 1,500 lb. per sq. in. of gross cross-sectional area as laid in the wall, and the compressive strength of any individual brick shall be not less than 1,000 lb. per sq. in. of gross cross-sectional area as laid in the wall.

4. The gross cross-sectional area of a brick shall be considered as the product of the length times the width of the unit as laid in the wall.

5. Concrete brick shall not absorb more than 12 per cent of the dry weight of the brick when tested as hereinafter specified except when they are made of concrete weighing less than 125 lb. per cu. ft. For brick made of concrete weighing less than 125 lb. per cu. ft., the average absorption in per cent by weight shall not be more than 12 multiplied by 125 and divided by the unit weight in pounds per cubic foot of the concrete under consideration.

6. Specimens for tests shall be representative of the commercial product of the plant.

7. Five specimens shall be required for each test.

8. The specimens used in the absorption test may be used for the strength test.

2. METHODS OF TESTING.

Absorption.

Absorption Test.—9. The specimens shall be immersed in clean water at approximately 70 deg. F. for a period of 24 hours. They shall then be removed, the surface water wiped off, and the specimens weighed. Specimens shall be dried to a constant weight at a temperature of from 212 deg. to 250 deg. F. and reweighed. Absorption is the difference in weight divided by the weight of the dry specimens and multiplied by 100.

Strength Test.

Strength Test.—10. Specimens for the strength test shall be dried to constant weight at a temperature of from 212 deg. to 250 deg. F.

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*This specification has been a tentative standard for a year. The convention, on February 26, 1925, adopted it to be sent to letter ballot as standard. Letter ballot was canvassed May 28, 1925.

11. The specimens to be tested shall be carefully measured for overall dimensions of length, width and thickness.

12. Bearing surfaces shall be made plane by capping with plaster of paris or a mixture of portland cement and plaster which shall be allowed to thoroughly harden before the test.

13. Specimens shall be accurately centered in the testing machine.

14. The load shall be applied through a spherical bearing block placed on top of the specimen.

15. Metal plates of sufficient thickness to prevent appreciable bending shall be placed between the spherical bearing block and the specimen.

16. The specimen shall be loaded to failure.

17. The compressive strength in pounds per square inch of gross cross-sectional area is the total applied load in pounds divided by the gross cross-sectional area in square inches.

Action on Standards.

The following specifications were adopted to be sent to letter ballot at the annual convention February 24-27, 1925. The ballots, canvassed May 28, 1925, indicate that all were adopted as Standards of the American Concrete Institute.

Standard Specifications for Concrete Drain Tile (P7B-25) (See Vol. 20, p. 678.)

Standard Specifications for Two-Course Portland Cement Concrete Pavement for Highways (S6B-25). (See Vol. 20, p. 70.)*

Standard Specifications for One-Course Portland Cement Concrete Street Pavement (S6C-25). (See Vol. 20, p. 716.)*

Standard Specifications for Two-Course Portland Cement Concrete Street Pavement (S6D-25). (See Vol. 20, p. 724.)*

Standard Specifications for Portland Cement Concrete Sidewalks (C2B-25). (See Vol. 20, p. 732.)

Standard Specifications for Concrete Building Block and Building Tile (P1A-25). (See Vol. 20, p. 663; also p. 602, this volume.)

Standard Specifications for Concrete Brick (P1B-25). (See Vol. 20, p. 666; also p. 604, this volume.)

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*Also see footnote on p. 537 of this volume.

A Letter from Sir E. Owen Williams, Builder of the Wembley Exposition.

5, S4, Georges Rd.,
Westminster,
S. W. 1.

27th January, 1925.

Dear Mr. Lindau:

I was gratified to receive your kind invitation on December 9th inviting me on behalf of your Institute to attend the annual Convention in Chicago at the end of February. I delayed replying as circumstances at that date did not permit me a definite decision. I received your cablegram, and with much regret I had to wire you my inability to accept your invitation.

I am responsible for the Amusement Park in the Paris Exhibition of this year, and in addition the British Empire Exhibition at Wembley is re-opening. Both Exhibitions are opening in April, and absence from England for the prolonged period necessary to visit you would be unfair in the extreme to my clients.

I can hardly express with what feelings I would have met in the flesh, fellow workers of another continent, with whom I am daily in contact in the spirit through the medium of the common material in which we work. Workers in the same medium must always be kindred, but workers in a material in its nursery days with all its problems and difficulties to solve are truly brothers.

In all progress there is lag between demand and supply. Industrial conditions have made big demands on the building industry. Nature has provided concrete to meet these demands. The world hesitatingly accepts the new material. This is but the inevitable lag, and I am confident we may look forward to a future in which concrete is the predominant material.

I wish your convention every success, and at some future date hope to meet you.

Sincerely yours,
E. Owen Williams.

A. E. Lindau, Esq.,
President,
American Concrete Institute,
Detroit, U. S. A.

AMERICAN CONCRETE INSTITUTE
BUSINESS REPORTS

Annual Report of the Board of Direction to the Members of the American Concrete Institute as of February 1, 1925.

Perhaps the best barometer of the progress of the Institute in the convention year just closing is in the increased membership, not only in this country but abroad, including all classes of workers interested in the solution of the technical problems in the uses of cement.

While the Institute's fiscal year ends June 30, it is interesting to consider the growth as between conventions. Between Feb. 1, 1924, and Feb. 1, 1925, our membership shows a net increase of 264. A summary of membership progress in the last five years is as follows:

Feb. 1, 1920	Membership	428
Feb. 1, 1921	Membership	627
Feb. 1, 1922	Membership	806
Feb. 1, 1923	Membership	981
Feb. 1, 1924	Membership	1,161
Feb. 1, 1925	Membership	1,425

(In the month of February, 1925, 117 applications were received for active membership and 5 for supporting membership; in March, 39 for active membership and 1 for supporting membership with losses by resignation and death which bring our membership April 1, 1925, to 1,469 active and 110 supporting, a total of 1,579.)

Obviously the responsibility of the Institute increases with the number who look to the organization for technical help in the field of concrete. With that in view, it will be the policy of the directors to make use of every practical means to keep in touch with the desires of the membership as to problems to be studied and to report as promptly as possible through individual papers and committee reports not only the results of technical research, but in addition (realizing the double responsibility of the organization) to see that the Institute *Proceedings* record the progress in the application to practical work of this knowledge developed through research.

Our twenty-first annual convention as it will be recorded in the *Proceedings*, Volume 21, constitutes the best report of the technical activities of the Institute and of the scope of the work being done. It is obvious that

this work must, through the Advisory Committee's activities, be so correlated and directed as to keep our convention program more and more open to the discussion of current problems of good practice.

With the increased membership in both active and supporting classes, the financial resources of the organization have increased and the financial condition of the organization is satisfactory as is indicated in the report of the auditor, Albert E. Horne, for the fiscal year ended June 30, 1924, which is appended.

A much larger number of supporting members is desirable, in view of the fact that active membership dues are scarcely sufficient to meet the increased publication and overhead expenses entailed by the increased work involved, so that any new work to be undertaken by the organization is largely dependent upon increase in supporting memberships.

ALBERT E. HORNE
PUBLIC ACCOUNTANT
115 HAGUE AVENUE
DETROIT

DETROIT, MICH., July 28, 1924.

Mr. Harvey Whipple, Treasurer,
American Concrete Institute,
Detroit, Michigan.

DEAR SIR:

In accordance with your instructions, I have made an examination of the books and records of the American Concrete Institute for the period from July 1, 1923, to June 30, 1924, and find all in order.

The results are set forth in accompanying statements marked Exhibits "A," "B," and "C."

The Cash in Bank amounting to $2,535, as shown by the Cash Book, was verified by reconciliation with the statement rendered by your bank as of June 30, 1924. During the examination paid checks were seen for all disbursements.

The remaining items in the statement of condition are shown in accordance with the records and have not been further verified.

The statement of the condition of Exhibit "A" is in accordance with the records and in my opinion shows the true financial condition of the Institute as of June 30, 1924.

Respectfully submitted,

(Signed) A. E. HORNE.

EXHIBIT "A."

AMERICAN CONCRETE INSTITUTE.

STATEMENT OF CONDITION.

June 30, 1924.

ASSETS.

Cash:		
In Bank	$2,535.00	
Imprest Cash	300.00	
Total Cash		$2,835.00
U. S. Treas. Certificates		7,500.00
Accounts Receivable:		
Active Members—146 @ $10.00; 3 @ $20.00	$1,520.00	
Contributing Members—3 @ $50.00	150.00	
Miscellaneous	195.29	
Total		1,865.29
Inventories:		
Proceedings Prior to 1919—418 @ $0.50	$209.00	
Proceedings, 1919-1922—474 @ $1.00	474.00	
Proceedings, 1923—158 @ $3.50	553.00	
Journals, 1914-15—452 @ $0.10	45.20	
Total		1,281.20
Total Assets		$13,481.49

LIABILITIES.

Deferred:		
Printing Proceedings, 1924 (Est.)	$5,300.00	
Dues Paid in Advance	390.00	
Reserve:		
For Loss of Delinquent Members	1,200.00	
Surplus	6,591.49	
Total Liabilities		$13,481.49

EXHIBIT "B."
AMERICAN CONCRETE INSTITUTE.
CASH RECEIPTS AND DISBURSEMENTS.
July 1, 1923, to June 30, 1924.
As Shown by Cash Book.

RECEIPTS.

Cash on hand June 30, 1923		$4,513.46
Certificates		3.00
Dues, Active		11,370.50
Dues, Contributing		5,850.00
Preprint Sales		527.88
Proceedings		1,392.72
Interest Earned		352.80
U. S. Treas. Certificates		1,000.00
20th Anniversary Fund		2,995.35
Total Receipts		$28,005.71

DISBURSEMENTS.

Certificates	$0.75	
Convention	846.31	
Exchange	13.12	
U. S. Treas. Certificates	5,500.00	
Nat'l Fire Protection Ass'n	60.00	
Audit	67.50	
Bond, Sec'y-Treas.	50.00	
Interest Paid	99.78	
Misc. Expense	146.35	
Preprints	1,550.70	
Office Expense	487.21	
Postage	660.46	
Printing, Multigraphing, Stationery and Sales Letters	2,600.25	
Proceedings	4,239.68	
Rent	379.92	
Salaries	5,309.08	
Traveling Expense	240.92	
Underwriters Lab.	272.51	
20th Anniversary Fund	2,995.35	
	$25,519.89	
Less—Discount Earned	49.18	
Total Disbursements		$25,470.71
Cash in Bank June 30, 1924		$2,535.00

EXHIBIT "C."

AMERICAN CONCRETE INSTITUTE.

Reconciliation of Bank Account.

June 30, 1924.

Balance July 1, 1923		$4,513.46
Receipts July 1, 1923, to June 30, 1924		23,622.04
Total		$28,135.50
Disbursements from July 1, 1923, to June 30, 1924		25,600.50
Ledger Balance June 30, 1924		$2,535.00
Add—Checks Outstanding:		
No. 983	$28.35	
No. 984	33.00	
No. 985	28.69	
		90.04
Bank Balance as per Statement June 30, 1924		$2,625.04

LIST OF REGISTRANTS.

An asterisk (*) designates a member.

*ABRAMS, DUFF A., 1951 Madison St., Chicago, Ill.
*ADAMSON, R., Hydro Electric Power Com. of Ont., 60 Forest Rd., Toronto, Can.
*AHLERS, JOHN G., Barney-Ahlers Const. Corp., 110 W. 40th St., New York, N. Y.
AIKEN, R. J., Aiken & Inness, Box 550, St. Catherines, Ont., Can.
ALBRECHT, RALPH W., 125 Reservoir St., Milwaukee, Wis.
ALLEN, J. M., Salt Lake City, Utah.
*ALLEN, LESLIE H., Hawthorne Roofing Tile Co., 2136 S. 48th Ave., Cicero, Ill.
*ALLEN, W. D. M., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*ANDERSON, LOUIS, JR., Alpha Portland Cement Co., Easton, Pa.
ARMS, LEO M., Railways Bureau, Portland Cement Assn., Chicago, Ill.
*ATWATER, RALPH W., McClellan & Junkersfeld, 68 Trinity Place, New York, N. Y.
*BARTHOLOMEW, TRACY, Mellon Institute, Pittsburgh, Pa.
*BARITELL, GEO. S., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
BATES, M. R., Fuel Mason Supply Dealer, Westmont, Ill.
BATES, P. H., U. S. Bureau of Standards, Washington, D. C.
*BAUER, E. E., University of Illinois, 205 Highway Lab., Urbana, Ill.
*BECK, ADAM L., Indiana Portland Cement Co., Indianapolis, Ind.
*BEEBY, FRANK F., Cement Gun Constr. Co., 537 S. Dearborn St., Chicago, Ill.
BEGGS, NORMAN, Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
BENO, JOSEPH P., 1239 Circle Ave., Forest Park, Ill.
BENTZ, C. L., Sanitary District of Chicago, 4415 West End Ave., Chicago, Ill.
*BERGER, P. S., Priester Constr. Co., 1006 Kahl Bldg., Davenport, Iowa.
*BERNIER, NAPOLEON M., 411 Walden St., Cambridge, Mass.
*BERNS, MAX A., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
BURDACK, WARREN T., Rock Falls, Ill.
BRIEN, M. J., 207 E. Ohio St., Chicago, Ill.
*BJOINDAHL, RICHARD, 175 19th Ave., Moline, Ill.
BLAINE, ETHEL E., Asst. Sec'y, American Concrete Institute, 1807 E. Grand Blvd., Detroit, Mich.
*BODIMER, EUG., Massey Concrete Products Corp., Peoples Gas Bldg., Chicago, Ill.
*BOGUE, ROBERT H., Portland Cement Assn., Bureau of Standards, Washington, D. C.
*BOURNE, C. L., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*BOYAJOHN, H. M., Boyajohn & Barr, 503 Brunson Bldg., Columbus, O.
*BOYER, EDWARD D., Atlas Portland Cement Co., 25 Broadway, New York, N. Y.
*BRADSHAW, GEORGE L., Independent Block & Cement Co., 2102 S. Harding, Indianapolis, Ind.
*BRAGGER, E. Y., Sandusky Cement Co., Cleveland, O.
*BRANDTZAEG, A., University of Illinois, 808 W. Indiana Ave., Urbana, Ill.
*BRASSERT, WALTER O., Michigian Silo Co., Bloomfield, Ind.
BRENNAN, MARTIN H., 1426 E. 66th Place, Chicago, Ill.
BREWER, R. D., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*BRICKETT, EDWARD M., Univ. of Ill., No. 300 L. A. M., Urbana, Ill.
*BROCKETT, R. M., C. A. Brockett Cement Co., 2035 E. 19th St., Kansas City, Mo.
*BROCKWAY, R. R., 616 6th St., So. East., Bridge Dept., N. P. Ry., Minneapolis, Minn.

*Broker, A. E., Badger Cement Tile Co., Plymouth, Wis.
Brown, J. F., Illinois Steel Co., 3426 E. 89th St., Chicago, Ill.
*Brundage, Avery, 110 S. Dearborn St., Chicago, Ill.
*Buente, C. F., Concrete Products Co. of America, Pittsburgh, Pa.
*Burns, D. F., Galien Concrete Tile Co., Fort Dearborn, Mich.
*Campbell, S. A., Ridge Construction Corp., 251 Elmdorf Ave., Rochester, N. Y.
Copoach, M. E., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
Carey, Bert & Co., Circle Ave., 7 14th St., Forest Park, Ill.
*Carey, W. H., Carey Concrete Co., Wisconsin Rapids, Wis.
Carlson, Carl V., Victor Carlson & Sons, 534 Sec. Bldg., Minneapolis, Minn.
*Calumet Steel Co., 208 S. La Salle St., Chicago, Ill. (A. S. Hook and F. G. Carrel.)
*Catlett, Richard H., Security Cement & Lime Co., Hagerstown, Md.
Cauffman, Geo. R., Stopher-Hildebrand Co., 441 E. Colfax Ave., So. Bend, Ind.
*Chubb, Jos. H., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*Clark, Robert J., Hinchley, Ill.
*Clemmer, H. F., 100 E. Washington St., Springfield, Ill.
*Christian, H. A., The New Jersey Zinc Co., Palmerton, Pa.
Clarke, H. V., The Philip Carey Co., 3611 Loomis Place, Chicago, Ill.
Cleve, Albert, Structural Materials Research Lab., 1951 W. Madison Ave., Chicago, Ill.
*Clousing, Louis, 213 City Hall, Minneapolis, Minn.
*Colburn, D. S., Marquette Cement Mfg. Co., Chicago, Ill.
*Collins, D. R., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*Condron & Post, 53 W. Jackson Blvd., Chicago, Ill. (T. L. Condron.)
*Cox, J. H., Grand Central Station, Chicago, Ill.
*Crabbe, Austin, The Cement Products Co., Davenport, Iowa.
Crabtree, D. L., J. W. Mueller & Co., Richmond, Va.
*Craig, H. N., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
Craig, James J., City of Zion, Ill.
Crandall, B. A., Sodus, Mich.
*Crane, Theodore, Yale University, New Haven, Conn.
Crepps, R. B., Prof. Testing Materials, Purdue University, 24 N. Salisbury St., W. Lafayette, Ind.
*Cross, Hardy, Prof. of Structural Eng., University of Ill., Urbana, Ill.
*Curtis, A. J. R., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
Dant, Carlton, 706 Greenleaf Ave., Wilmette, Ill.
Davey, B. C., U. S. Materials Co., 1551 Kingsbury, Chicago, Ill.
*Davis, E. E., 608 S. Dearborn St., Chicago, Ill. (E. E. Davis Co.)
*Dayton, K. P., Universal Portland Cement Co., 208 S. La Salle St., Chicago, Ill.
Davis, H. L., I. C. R. R., 1769 Wilson Ave., Chicago, Ill.
Davis, R. C., 906-226 W. Jackson St., Chicago, Ill.
De Berard, W. W., Western Editor, *Engineering News-Record*, 1570 Old Colony Bldg., Chicago, Ill.
*Deinboll, F. K., N. Y. C. R. R., Cleveland, O.
Deuchler, Walter E., 366 Garfield Ave., Aurora, Ill.
*Deutsch, M. J., Buffalo Wash Tray Works, Buffalo, N. Y.
*Dockstader, E. A., Stone & Webster, Inc., 147 Milk St., Boston, Mass.
*Doll, Herman W., Superior Cement Stone Co., Bedford, O.
*Douglass, A. S., The Detroit Edison Co., 2000 2nd Blvd., Detroit, Mich.
*Douthett, C. L., Humboldt Gravel & Tile Co., Humboldt, Iowa.
*Dowdall, E. J., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
Dudgeon, O. A., Westmont, Ill.
*Dugan, Chas. B., National Steel Fabric Co., 1118 Straus Bldg., Chicago, Ill.
Dundas, F. C., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*Dunnells, Clifford A., Carnegie Institute of Technology, Pittsburgh, Pa.

*Durant, Grant J., Froehling & Robertson, Inc., 815 E. Franklin St., Richmond, Va.
*Durgin, Frank L., Jr., Cramp & Co., 801 Denckla Bldg., Philadelphia, Pa.
*Dunn, J. A., Independent Concrete Pipe Co., 201 N. West St., Indianapolis, Ind.
Duwe, Edward, Oshkosh, Wis.
*Earley, John J., 2131 G St., N. W., Washington, D. C.
Eaton, G. S., General Educational Bureau, Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
Elursold, Fred H., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
Eckert, R. T., Structural Materials Research Lab., 2524 N. Spaulding Ave., Chicago, Ill.
*Ege, C. R., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*Egelhoff, R. T., Turner Constr. Co., 11 Goodell St., Buffalo, N. Y.
Elwell, J. S., National Lime Assn., 844 Rush St., Chicago, Ill.
*Enblom, Albert, Nelson-Enblom Co., 917 Plymouth Bldg., Minneapolis, Minn.
*Emerson, F. H., Celite Products Co., 11 Broadway, New York, N. Y.
*Fairchild, L. F., Eastman Kodak Co., 3195 Lake Ave., Rochester, N. Y.
*Farmer, Homer G., Universal Portland Cement Co., 526 Frick Bldg., Pittsburgh, Pa.
*Faulkner, H. F., City Engineering Dept., Seattle, Wash.
*Ferguson, John A., 1419 N. Euclid Ave., Pittsburgh, Pa.
*Felsheim, S. W., The Master Builders Co., Cleveland, O.
Flodin, H. L., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*Forrest, V. E., Bland Engineering Co., Minneapolis, Minn.
*Forshee, W. A., Wells Bros. Const. Co., 914 Monadnock Bldg., Chicago, Ill.
Foshinbaur, V. G., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*Foster, C. B., Foster Eng. Service Co., Indianapolis, Ind.
*Fournie, Arthur J., R. R. No. 3, Belleville, Ill.
Fournie, C., Swansea Stone Works, R. R. No. 3, Belleville, Ill.
Fournie, Fred, Swansea Stone Works, Belleville, Ill.
Franks, C. D., Portland Cement Assn., 1825 Conway Bldg., Chicago, Ill.
*Freeman, J. Edgar, Par Lock Appliers of Chicago, 122 S. Michigan Ave., Chicago, Ill.
Freund, C. S., Continental Cement Tile Co., Straus Bldg., Chicago, Ill.
*Fritz, Peter, Buffalo Wash Tray Works, Buffalo, N. Y.
*Fullenwider, C. V. R., The Philip Carey Co., Locklan, Cincinnati, O.
*Gaston, H. F., W. E. Dunn Mfg. Co., Holland, Mich.
*Ginder, J. W., Office, Supervising Architect, Treasury, Washington, D. C.
*Ginsberg, Frank J., Chain Belt Co., 18 E. 41st St., New York, N. Y.
*Glose, Robert L., National Steel Fabric Co., Union Trust Bldg., Pittsburgh, Pa.
Going, H. C., Going Road Machinery Co., 1624 1st Ave., Birmingham, Ala.
*Goldie, Wm., Jr., Goldie Mfg. Corp., 106 Biddle Ave., Pittsburgh, Pa.
*Gonnerman, H. F., Portland Cement Assn., 1951 W. Madison St., Chicago, Ill.
*Goodwin, Paul M., The Solvay Process Co., 112 W. Adams St., Chicago, Ill.
Gosswein, O. H., Universal Portland Cement Co., Chicago, Ill.
*Gould, Harley J., The Ferro Concrete Constr. Co., 3rd and Elm Sts., Cincinnati, O.
*Grady, J. C., Turner Constr. Co., 178 Tremont St., Boston, Mass.
Graham, Raymond J., H. L. Stevens & Co., 30 N. Michigan Ave., Chicago, Ill.
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Haegert, L. V., A. T. & S. F. Ry., 2240 W. 8th St., Topeka, Kan.
Haegert, L. V., Mrs., Topeka, Kan.
Haggander, G. A., 547 W. Jackson Blvd., Chicago, Ill.
Hale, C. D., Portland Cement Assn., 1537 Conway Bldg., Chicago, Ill.
Hammerschmidt, Geo. F., Elmhurst Chicago Stone Co., Elmhurst, Ill.

HAMMERSCHMIDT, MARTIN, Elmhurst Chicago Stone Co., Elmhurst, Ill.
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*HITCHCOCK, FRANK A., Bureau of Standards, Washington, D. C.
HODUETT, R. M., Chicago Board of Education, 8014 Jeffery Ave., Chicago, Ill.
*HOLMGREN, ROBERT W., R. E. Schmidt, Garden & Martin, 104 S. Michigan Ave., Chicago, Ill.
HOLLMGSHRAD, T. H., Zion Inst. & Ind., Zion, Ill.
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*HUNDHAUSEN, CARL, Wm. Hundhausen, 421 Barker Ave., Peoria, Ill.
*HUNDHAUSEN, 421 Barker Ave., Peoria, Ill.
HUSSEY, J. W., Atlas Portland Cement Co., 134 S. La Salle St., Chicago, Ill.
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*HUXHOLD, P. F., 1127 Garfield St., Oak Park, Ill.
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*IRWIN, A. C., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*JACKSON, FRANK H., Bureau of Public Roads, Washington, D. C.
*JAHNE, EMIL R., Perrysville, Ind.
JAMES, J. R., Detroit Edison Co., 2000 2nd Ave., Detroit, Mich.
JASSON, WM. A., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
JENNINGS, D. F., *Concrete*, 1807 E. Grand Blvd., Detroit, Mich.
JOHNSON, C. L., Atlas Portland Cement Co., Chicago, Ill.
*JOHNSON, NATHAN C., 342 Madison Ave., New York, N. Y.
*JOHNSON, VIRGIL L., Board of Education, Philadelphia, Pa.
*JONES, W. C., Roseland Concrete Products Co., 12110 Michigan Ave., Chicago, Ill.
*JORDAN, M. K., Federal Cement Tile Co., Westminster Bldg., Chicago, Ill.
KAISER, W. G., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
KASANKE, PAUL L., 1335½ 34th St., Milwaukee, Wis.
KELLY, J. W., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
KIMMEL, W. D., Portland Cement Assn., 1659 Pratt Blvd., Chicago, Ill.
*KINNEY, WM. M., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*KIRCHER, PAUL, Massey Concrete Products Corp., 968 Peoples Gas Bldg., Chicago, Ill.
*KONSTANT, NICHOLAS Z., 1473 Carmen Ave., Chicago, Ill.

KRIST, O. H., A. W. French & Co., 208 N. Clinton St., Chicago, Ill.
KROHNE, FRANK C., 495 21st Ave., Milwaukee, Wis. (Prestone Co.)
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KUESTEN, E. F., Roseland Concrete Products Co., 12110 S. Michigan Ave., Chicago, Ill.
*KVITRUD, I., 754 Builders Exchange, Minneapolis, Minn.
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LAUGHTON, P. W., 4551 Kingsbury Ave., Chicago, Ill.
*LAPHAM, JOHN R., Geo. Washington University, 807 Quackenbas St., N. W., Washington, D. C.
LARSON, CARL T., 749 14th St., Moline, Ill.
LARSON, L. J., University of Ill., 210 Engineering Hall, Urbana, Ill.
LAUCKS, H. F., Ideal Concrete Mach. Co., 1374 Wrigley Bldg., Chicago, Ill.
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*LEFFLER, R. R., Sanitary District, 7021 Oriole Ave., Chicago, Ill.
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*MCCULLOUGH, F. M., Carnegie Institute of Technology, Pittsburgh, Pa.
*MCFALL, H. C., Atlas Portland Cement Co., 134 S. La Salle St., Chicago, Ill.
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*MCMILLAN, F. R., Structural Materials Research Lab., Lewis Inst., Chicago, Ill.
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MARTIN, S. G., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
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*MEADE, S. J., The Heine Chimney Co., 111 W. Washington St., Chicago, Ill.
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*MENKE, E. W., Bates Valve Bag Co., 8200 S. Chicago Ave., Chicago, Ill.
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RIDDEWOLD, A. S., Schmidt Garden & Martin, 1515 Monroe Bldg., Chicago, Ill.
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STILEX, D. H., The American Bureau of Inspection & Tests, 1741 Monadnock Block, Chicago, Ill,
*STONE, O. O., Portland Cement Assn., Dime Bank Bldg., Detroit, Mich.
*STRAUB, F. J., 1207 Victoria Ave., New Kensington, Pa.
*SWANSON, A. G., Omaha Concrete Stone Co., Omaha, Neb.
SWETT, LESLIE W., Lakewood Engineering Co., 11 S. La Salle St., Chicago, Ill.
*TALBOT, A. N., University of Ill., Urbana, Ill.
*TALBOT, K. H., 52 S. State St., Elgin, Ill.
TAPP, ROSCOE H., Herman W. Tapp, 920 Delaware Ave., Fort Wayne, Ind.
*THOMSON, W. S., Kalman Steel Co., 1482 Wrigley Bldg., Chicago, Ill.
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*TOBIAS, M. C., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
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VERHOEVEN, CLARENCE G., R. C. P. Co., 11507 Michigan Ave., Chicago, Ill.
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*WALKER, E. H., City of Rochester, N. Y.
WALLACE, ROBERT W., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
WALKER, FRANK R., 536 Lake Shore Drive, Chicago, Ill.
*WALKER, STANTON, Structural Materials Research Lab., Lewis Institute, Chicago, Ill.
*WARD, A. A., Ward-Kurz Co., 415 E. State St., Ithaca, N. Y.
*WARD, H. A., Turner Constr. Co., 71 Norwood Ave., Buffalo, N. Y.
*WARREN, G. E., Portland Cement Assn., 1537 Conway Bldg., Chicago, Ill.
*WARREN, SAMUEL, Atlas Portland Cement Co., 25 Broadway, New York, N. Y.
*WATSON, T. P., Engineering Dept., P. R. R., 1155 Liberty Ave., Pittsburgh, Pa.
WEBB, C. A., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
WEBSTER, ALBERT E., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
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*WHIPPLE, HARVEY, *Concrete,* 1807 E. Grand Blvd., Detroit, Mich.
*WHIPPLE, HARVEY, MRS., Detroit, Mich.
WHITFIELD, A. G., 611 Fulton St., Chicago, Ill.
*WIGHT, FRANK C., *Engineering News Record*, New York, N. Y.
WILEY, JAMES A., P. R. R., New Brighton, Pa.
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WILLIAMS, H. R., Material Service Corp., 21 N. La Salle St., Chicago, Ill.
*WILLIAMS, ROBERT L., Luzerne County Road & Bridge Dept., Wilkes-Barre, Pa.
*WILSON, E. B., American Bureau of Inspection & Tests, 53 W. Jackson St., Chicago, Ill.
WILSON, J. K., Ceresit Waterproofing Corp., 10 S. La Salle St., Chicago, Ill.
*WILSON, J. P., Dept. of Bldgs. & Safety Engineering, City of Detroit, 8026 Moffatt St., Detroit, Mich.
WILSON, RAYMOND, Portland Cement Assn., 1951 W. Madison Ave., Chicago, Ill.
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WILSON, W. A., Pit and Quarry, 538 S. Clark St., Chicago, Ill.
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WINSTEAD, R. D., *American Contractor,* 131 N. Franklin St., Chicago, Ill.
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*WOOLSON, IRA H., National Board of Fire Underwriters, 76 William St., New York, N. Y.

WOODWORTH, P., M. S. M. R. L., 791 Crescent Blvd., Glenellyn, Ill.
WOODWORTH, P. M., S. M. R. L., 791 Crescent Blvd., Glenellyn, Ill.
WRAY, THEODORE S., Pennsylvania Railroad, Polk Street Freight Terminal, Chicago, Ill.
WRIGHT, H. S., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*YANDELL, C. H., Yandell & Conger, Jackson, Tenn.
YORK, E. L., I. C. R. R. Co., 7421 Bennett Ave., Chicago, Ill.
*YOUNG, ROY N., Lehigh Portland Cement Co., Allentown, Pa.
*YOUNG, R. B., Hydro-Electric Power Comm., Toronto, Ont., Can.
ZIEGLER, CLEM. M., Winn Lumber Co., Mt. Vernon, Ill.
ZOEHER, C. A., 2023 Sycamore St., Milwaukee, Wis.

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Abe, Mikishi, 24 Mita S Temple, Shiba, Tokyo, Japan.

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Abrams, Duff A., Lewis Institute, 1951 W. Madison St., Chicago, Ill.

Acheson, W. M., Div. Engr., New York State Highway Commission, Syracuse, N. Y.

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Allen, Leslie H., 2137 S. 48th St., Cicero, Ill. (Concrete Tile Mchry Co.)

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COPPERSTONE PRODUCTS CO., THE, 220 Albion St., Toledo, Ohio. (A. A. Bennett.)

CORBEN, H. J., City Hall, Darling St., Cape Town, Cape Province, South Africa.

CORBETT, ALEXANDER JOHN, "Pacific House," 249 George St., Sydney, N. S. W., Australia. (Clandulla Portland Cement Co.)

CORE JOINT CONCRETE PIPE CO., INC., Irvington, N. J. (V. F. Amar.)

CORIMER, ERNEST, Drummond Bldg., Montreal, Canada.

CORRIDON, JOSEPH B., 1625 Arch St., Philadelphia, Pa. (Stone & Webster, Inc.)

CORY, RUSSEL G., 50 Church St., New York, N. Y.

C. & S. CONCRETE PRODUCTS CO., P. O. Box 2222, St. Petersburg, Fla. (L. E. Carpenter.)

COWELL LIME & CEMENT CO., HENRY, 2 Market St., San Francisco, Calif.

COWPER CO., THE JOHN W., Fidelity Bldg., Brooklyn, N. Y. (J. W. Cowper.)

CRAIG-CURTISS CO., THE, 4614 Prospect Ave., Cleveland, Ohio.

*CRAMP & CO., 801 Denckla Bldg., Philadelphia, Pa. (F. L. Durgin, Jr.)

CRANDALL ENGINEERING CO., THE, 102 Borner St., East Boston, Mass. (J. Stuart Crandall.)

CRANE, THEODORE, Yale University, New Haven, Conn.

CRANFORD, FREDERICK L., INC., 149 Remsen St., Brooklyn, N. Y. (F. L. Cranford.)

CRARY, ALEX P., 1958 Bogart Ave., Borough of Bronx, N. Y. (Thompson & Binger, Inc.)

CREBE, W. G., 949 Scribner St., Grand Rapids, Mich.

*CRESCENT PORTLAND CEMENT CO., Wampum, Pa. (David M. Kirk.)

CROOKS, SAMUEL IRWIN, JR., Maungawhau Road, Khyber Pass, Auckland, New Zealand.

CROSS, HARDY, Prof. of Structural Engineering, University of Illinois, Urbana, Ill.

CROWN SIDEWALK & BLOCK CO., INC., 2500 Washington St., N. E., Minneapolis, Minn. (Harry A. Anderson.)

*CROZIER-STRAUB, INC., 102 W. 42nd St., New York, N. Y. (G. Edgar Allen.)

CRUM, R. W., Iowa State Highway Commission, Ames, Iowa.

CUMMINGS, A. E., Room 1912, 111 W. Monroe St., Chicago, Ill. (Raymond Concrete Pile Co.)

CUMMINS, CHARLES A., 243 Calvert Bldg., Baltimore, Md. (Consolidated Engineering Co.)

CURTIS, A. J. R., 111 W. Washington St., Chicago, Ill. (Portland Cement Assn.)

DAHLBERG, B. G., 645 Michigan Ave., Chicago, Ill. (The Celotex Co.)

DAIDONE, ANTHONY J., 6114 16th Ave., Brooklyn, N. Y. (Atlas Art Stone Works.)

DAMON, ALBERT F., JR., P. O. Bldg., Upper Darby, Del. Co., Pa.

DANAHER, W. E., 11 Goodell St., Buffalo, N. Y. (Turner Construction

DANAHER, W. E., 111 Goodell St., Buffalo, N. Y. (Turner Construction Co.)

DARLING, E. E., 47 Home Bank Bldg., Hamilton, Ont.

DARUVALA, J. P., 70 Trinity St., Dhobi Talao, Bombay, India.

DAVIDSON, JOHN M., 216 N. 3rd St., Morgan Park, Duluth, Minn.

DAVIS, A. P., 505 Santa Ray Ave., Oakland, Calif.

DAVIS, B. H., 17 Battery Place, New York City. (Bureau of Standards.)

DAVIS, E. E., 1540 Transportation Bldg., Chicago, Ill. (E. E. Davis Co.)

DAVIS, FREDERICK C., 515 Buena Vista Ave., San Francisco, Calif.

DAVIS-HELLER-PEARCE CO., Delta Bldg., Stockton, Calif. (H. Y. Davis.)

DAVIS, HERBERT A., 1662 Park Rd., Washington, D. C. (Washington Concrete Products Corp.)

DAVIS, WATSON, 1442 Rhode Island Ave., N. W., Washington, D. C.

DAVISON, R. GLENN, Jamesburg, N. J.

DAW, E. A. H., Somerset House, 9 Martin Place, Sydney, Australia. (Expanded Steel & Concrete Products Co.)

DAWES, EDWIN ALLEN, c/o Dawes Constr. Supply Co., Thomasville, Ga.

DAWSON, F. M., Lawrence, Kan. (University of Kansas.)

DAY, W. P., California Comm. Union Bldg., San Francisco, Calif.

DEEDY, WALTER E., 901 Mills Bldg., El Paso, Texas.

DEFRAIN SAND CO., 804 Finance Bldg., Philadelphia, Pa. (J. L. Durnell.)

DEINBOLL, F. K., 1263 Brockley Ave., Lakewood, Ohio.

DELEUW, C. E., 111 W. Washington St., Chicago, Ill. (Kelker, DeLeuw & Co.)

DELL, ROBERT B., 103 S. 6th St., Duquesne, Pa. (City Engineer.)

DENSMORE, LECLEAR & ROBBINS, 88 Broad St., Boston, Mass. (Henry C. Robbins.)

DENTON, ARTHUR P., 785 Market St., San Francisco, Calif. (Dist. Engr., Portland Cement Assn.)

DENTON & CO., 7 E. 42nd St., New York City. (P. E. Eisenmenger.)

DENTON, W. EDWARD, 3809 Alton Place, N. W., Washington, D. C.

DEPARTMENT OF CIVIL ENGINEERING, University of Texas, Austin, Texas. (Dean T. U. Taylor.)

DEPARTMENT OF THE INTERIOR, San Juan, Porto Rico. (Commissioner Guillermo Esteves.)

DESLAURIERS COLUMN MOULD CO., INC., 233 Broadway, New York, N. Y. (Henry A. Dahlen.)

Destremps, Louis G., 49 Borden Blk., Fall River, Mass.
Detroit Edison Co., Detroit, Mich. (A. S. Douglas, Chief Engineer.)
Detroit Edison Co., 2000 Second Ave., Detroit, Mich. (M. A. Carabin, Librarian.)
Detroit Public Library, Woodward and Kirby Aves., Detroit, Mich. (Adam Strohm, Librarian.)
Dewey, George F., 613½ Ohio St., Cairo, Ill.
Dewey Portland Cement Co., Suite 301 Mutual Bldg., Kansas City, Mo. (F. E. Tyler.)
Dexter Portland Cement Co., Nazareth, Pa. (J. B. Brobston.)
*Dexter Portland Cement Co., 350 Madison Ave., New York, N. Y. (R. W. Hilles.)
*Dexter Portland Cement Co., Nazareth, Pa. (Joseph Brobston.)
Diamond Concrete Products Co., 42nd and Parker Sts., Omaha, Neb. (Frank Whipperman, Pres.)
Dickerson, John H., 139 Rector St., Elizabeth, N. J.
Dieckmeyer, L. M., 4909 Parkview Place, St. Louis, Mo.
Dillingham, George H., 1734 Spruce St., Philadelphia, Pa.
Dingman, Chas. F., 15 Grove St., Palmer, Mass. (Palmer Constr. Co., Inc.)
Di Stasio, Joseph, 136 Liberty St., New York City. (J. Di Stasio & Co.)
Dittmyer, Alex, 1213 Cora St., Joliet, Ill.
Diver, M. L., San Antonio, Texas.
Dixie Concrete Products Co., 1010 James Bldg., Chattanooga, Tenn. H. B. Springer.)
*Dixie Portland Cement Co., Chattanooga, Tenn. (Richard Hardy, Pres.)
*Dixie Portland Cement Co., Chattanooga, Tenn. (Richard Hardy, Pres.)
Dixon, DeForest, 244 Madison Ave., New York, N. Y. (Turner Construction Co.)
Dockstader, Ernest A., 147 Milk St., Boston, Mass. (Stone & Webster.)
Doelman, Herman F., 507 N. Charles St., Baltimore, Md.
Doelman, R. E., 311 13th St., N. E., Washington, D. C.
Donald, G. C., c/o Davenport Duntile Works, 613-617 Harrison St., Davenport, Ia.
Donaldson, W. E., Rooms 261 and 262, Frick Annex, Pittsburgh, Pa. (Carnegie Steel Co.)
Donaldson & Meier, 1310 Penobscot Bldg., Detroit, Mich. (H. W. Meier.)
Donohue, Jerry, 608 N. 18th St., Sheboygan, Wis.
Doolittle, C. M., Dundas, Ont. (Canada Crushed Stone Corp., Ltd.)
Dow Chemical Co., The, Midland, Mich. (S. W. Putnam.)
Dowdell, C. O., 79 W. Washington St., Chicago, Ill.
Downer, Robert G., Runnemede, Camden Co., N. J.
Downs, Allan B., 61 Elm St., Lebanon, N. H.
Doyle, William T., 128 N. Wells St., Chicago, Ill.
Drehman Paving Co., 2622 Parrish St., Philadelphia, Pa. (C. E. Drehman.)

DRESSER-MINTON-SCOBELL CO., THE, 1082 The Arcade, Cleveland, Ohio. (J. H. Minton.)

DREW CO., INC., FRED, Woodward Bldg., Washington, D. C. (Fred Drew.)

DREYER, WALTER, 445 Sutter St., San Francisco, Calif. (Pacific Gas & Electric Co.)

DUBUC, N. P., JR., 1742 Ludlow Ave., Indianapolis, Ind.

DUCKETT, N. S., 12 Castlefield Ave., Toronto, Ont., Canada.

DUFFERIN CONSTRUCTION CO., LTD., Foot of Bathurst St., Toronto, Canada. (James Frances Chini.)
(James Franceschini.)

DUNLAP, R. M., 608 S. Dearborn St., Chicago, Ill. (Federal Cement Tile Co.)

DUNN, OSWALD T., 407 N. 6th St., Paducah, Ky.

DUNNELLS, CLIFFORD G., Head of Dept. of Bldg. Construction, Carnegie Inst. of Technology, Pittsburgh, Pa.

DUNSDON, A. C., c/o East Indian Railway House, 73 King William St., London, E. C. 4, England.

DUPUY, ALBERTO, Apartado 893, Bogota, Colombia, S. A.

DUQUESNE SLAG PRODUCTS CO., Diamond Bank Bldg., Pittsburgh, Pa. (C. L. McKenzie.)

DUTTON, C. B., Monadnock Block, Chicago, Ill. (Besser Sales Co.)

DWYER, JOHN R., Bureau of Standards, Washington, D. C.

EARLEY, JOHN J., 2131 G St., N. W., Washington, D. C.

EARLE, LTD., G. AND T., Wilmington, Hull, England. (G. F. Earle.)

EAST JERSEY CONCRETE PRODUCTS ASSN., Avon-by-the-Sea, N. J. (John Shapter, Secy.)

EASTMAN KODAK CO., Rochester, N. Y. (C. K. Flint.)

EBENER, FRITZ, Schonleinstrasse 3, Essen (Rheinland), Germany.

EDDY BROTHERS, Building Block, Manorville, Pa. (D. A. Eddy.)

*EDISON, THOMAS A., Orange, N. J.

EGBERT, GEORGE W., JR., 1521 Cortelyou Rd., Brooklyn, N. Y.

EGE, C. R., 111 W. Washington St., Chicago, Ill. (Portland Cement Assn.)

EGELHOFF, R. F., Turner Constr. Co., 11 Goodell St., Buffalo, N. Y.

EHLERT, E. H., 9119 Harvard Ave., Cleveland, Ohio.

EITZEN, HENRY R., 110 E. 42nd St., New York, N. Y. (Kalman Steel Co.)

EKHOLM, S. L., 896-902 Farrar St., Cadillac, Mich. (Helm Brick Machine Co.)

ELDRIDGE, H. W., New City, Rockland County, Y. Y. (Cement-Gun Co., Inc.)

ELFORD, H., 555 Park St., So., Columbus, Ohio.

ELK RIVER CONCRETE PRODUCTS CO., Elk River, Minn. (D. W. Longfellow.)

ELLOK CORPORATION, 956 Ellicott Sq., Buffalo, N. Y. (James G. Davis.)

EMLEY, WARREN E., 3705 Keokuk St., Washington, D. C. (Bureau of Standards.)

EMMANUELOV, V. E., Bombay Municipality, Office of Works, Love-Grove Road, Worli, Bombay, India.

ENGELMANN, L., Engr., 3644 33rd St., San Diego, Calif.

ENGINEER, R. K., 39 Marine Line, Fort Bombay, India.

ENGINEERING DEPT., City of Rochester, N. Y., 52 City Hall, Rochester, N. Y. (E. H. Walker.)

ENGINEERING EXPERIMENT STATION, University of Texas, Box L, University Station, Austin, Texas. (H. R. Thomas.)

ENGINEERING NEWS-RECORD, 10th Ave. at 36th St., New York City. (Frank C. Wight.)

ENGLAND, JOHN, Gibbs Chambers, Martin Place, Sydney, N. S. W., Australia.)

ENGSTROM & Co., 1117 Chapline St., Wheeling, W. Va. (J. W. Wynn.)

ESSELSTYN & CAREY, 603 Hofman Bldg., Detroit, Mich. (H. H. Esselstyn.)

*ETTINGER CONTRACTING CO., INC., 44 Court St., Brooklyn, N. Y. (Louis Ettinger.)

EUPHRAT-HANLEY ESTIMATING & ENGINEERING BUREAU, 305 Walnut St., Cincinnati, Ohio. (Hunter W. Hanley.)

EVANS, FRANK M., 10 W. 28th St., New York, N. Y.

EVERHART, C. C., 539 Van Buren St., Milwaukee, Wis.

EWING CONCRETE CO., DAVIS, 712 E. Empire St., Bloomington, Ill. (Davis Ewing, Pres.)

EY, VICTOR, N. 4th St., Woodside, Long Island, N. Y.

EYRICK, GEO. J., JR., 800 Marquette Bldg., Detroit, Mich.

FABRIQUE DE CIMENT PORTLAND ARTIFICIEL, Hemixem-Anvers, Belgium. (J. Van Den Heuvel.)

FAIR, T. R., Great Eastern Hotel, Calcutta, India.

FAIRCHILD, LEROY F., 3211 Lake Ave., Rochester, N. Y. (Eastman Kodak Co.)

FAIRMAN, J. R., 1805 Age Herald Bldg., Birmingham, Ala. (Dist. Engr., Portland Cement Assn.)

FARMER, HOMER G., Frick Bldg., Pittsburgh, Pa.

FARNHAM, R., Broad Street Station, P. R. R., Philadelphia, Pa.

FAULKNER, H. F., City Engineer's Dept., Seattle, Wash.

FAY, FREDERICK R., 200 Devonshire St., Boston, Mass. (Fay, Spofford & Thorndike.)

FEDERAL CEMENT TILE CO., 608 S. Dearborn St., Chicago, Ill. (Leland J. Wilhartz, Secy.)

FEDERAL CONCRETE CO., 667 Wyoming Ave., Buffalo, N. Y. (Walter E. Jones, Secy.)

FELDRAPPE, M. G., 1523 E. 81st St., Cleveland, Ohio. (A. A. Lane Construction Co.)

FERGUSON, JOHN A., 1419 N. Euclid Ave., Pittsburgh, Pa.

FERGUSON & CO., J. B., Hagerstown, Md. (J. B. Ferguson.)

*FERGUSON CO., JOHN W., United Bank Bldg., 152 Market St., Paterson, N. J. (John W. Ferguson.)

FERGUSON, LEWIS R., 1400 Land Title Bldg., Philadelphia, Pa. (Light, Hollister & Ferguson.)

FERRO BUILDING PRODUCTS CO., 1619 Grand Central Terminal, New York, N. Y. (A. Hamburger.)

*FERRO CONCRETE CONSTRUCTION CO., Third and Elm Sts., Cincinnati, Ohio. (W. P. Anderson.)

FERRO CONCRETE CONSTRUCTION CO., Third and Elm Sts., Cincinnati, Ohio. (H. D. Loring.))

FERRY CO., INC., JAMES, Virginia and Mediterranean Aves., Atlantic City, N. J. (James V. Ferry.)

FINLAY, L. G., 614 Union Bldg., Cleveland, Ohio. (Raymond Concrete Pile Co.)

FIRESTONE, SIEGMUND, 59-61 South Ave., Rochester, N. Y.

FISCHER, JR., ANDREW, 140 Cedar St., New York City. (c/o Walter Kidde & Co., Inc.)

FISCHER, L. J., 51 Wall St., New York City. (Thompson-Starrett Co.)

FISHER, EVAN THOMAS, 660 Market St., San Francisco, Calif.

FISHER, OTTO F., Vasagatan 38, Stockholm, Sweden. (Betongbyran.)

FISCHER-DEVORE CONSTRUCTION CO., 1025 Dixie Terminal Bldg., Cincinnati, Ohio. (Frank F. Fisher.)

*FISKE-CARTER CONSTRUCTION CO., 11 Foster St., Worcester, Mass. (Burton C. Fiske.)

FITTON, HARRY R., 542-N. Meridian St., Indianapolis, Ind. (Mothershead and Fitton.)

FLAM, STEPHEN, P. O. Box 265, Huntington Park, Calif.

FLEMING, GEO. S., 120 Madison Ave., Detroit, Mich. (Robert O. Derrick, Inc.)

FLETCHER, AUSTIN B., Cosmos Club, Washington, D. C.

FLETCHER, RICHARD G., 921 15th St., N. W., Washington, D. C. (Fletcher Fireproofing Co.)

FLETCHER-THOMPSON, INC., P. O. Box 85, Bridgeport, Conn. (Edward A. Lambert.)

FLIGHT, OSCAR, Carpenter St., Bendigo Victoria, Australia.

FOGG, RALPH J., Lehigh University, Bethlehem, Pa.

*FOOTE COMPANY, INC., THE, Nunda, N. Y. (F. L. Dake.)

FORD, MATT, Caldwell, Kan.

FORREST, V. E., Kennedy and L St., N. E., Minneapolis, Minn.

FOSTER, ALEXANDER, JR., Delaware Ave. and Marlborough St., Philadelphia, Pa. (West Jersey Sand and Supply Corp.)

FOSTER, C. B., 726 K. of P. Bldg., Indianapolis, Ind.

FOUGNER, HERMANN, 103 Park Ave., New York City. (Thompson & Binger, Inc.)

FRANCISCO, F. LEROY, 511 5th Ave., New York City. (Francisco & Jacobus.)

Frank, Harry H., 207 Fulton Bldg., Pittsburgh, Pa.
*Franklin Steel Works, Franklin, Pa. (E. E. Hughes.)
Franklin, J., 1218 Wrigley Bldg., Chicago, Ill. (Ideal Concrete Machinery Co.)
Fraser, Alexander, Department of Roads, Quebec, Canada.
Frauenfelder, Herman, 4600 Chippewa St., St. Louis, Mo.
Frech, H. E., 1313 Syndicate Bldg., St. Louis, Mo. (Dist. Engr., Portland Cement Assn.)
Freeland, Roberts & Co., 1212 Ind. Life Bldg., Nashville, Tenn. (M. S. Roberts.)
Freeman, John E., 122 S. Michigan Ave., Chicago, Ill.
Freeman, J. R., 815 Grosvenor Bldg., Providence, R. I.
Freeman, P. J., 519 Smithfield St., Pittsburgh, Pa.
French, A. W., 202 Russel St., Worcester, Mass. (Worcester Polytechnic Institute.)
French & Co., S. H., 4th and Callowhill Sts., Philadelphia, Pa. (F. T. McBride.)
Freund, I. H., 608 Dearborn St., Chicago, Ill. (Federal Cement Tile Co.)
Fridstein, Meyer, 1753 Conway Bldg., Chicago, Ill.
Friebele, J. F., Broad St. Bank Bldg., Trenton, N. J. (Karno-Smith Co.)
Friel, Francis S., c/o Albright & Mebus, 1502 Locust St., Philadelphia, Pa.
Friske, A. W., Alois P. O., Wis.
Froehling & Robertson, Richmond, Va. (H. C. Froehling.)
Frost, Chamberlain & Edwards, Slater Bldg., Worcester, Mass.
Fruchtbaum, J., 726 Genesee Bldg., Buffalo, N. Y. (Truscon Steel Co.)
Fruin-Colnon Contracting Co., 502 Merchants-Laclede Bldg., St. Louis, Mo. (A. P. Greensfelder, Secy.)
Fry, Lynn W., Office of State Architect, Ann Arbor, Mich.
Fuller & McClintock, 170 Broadway, New York City. (Geo. W. Fuller.)
Furber, Pierce P., 1209 Fairmont St., N. W., Washington, D. C.
Furlong, Irving, 81 Appraiser Bldg., San Francisco, Calif. (Bureau of Standards.)
Fusejima, Shinkuro, Engineering Dept., South Manchuria Railway Co., Ryusan, Chosen, Japan.
Gabriel Steel Co., 1150 Penobscot Bldg., Detroit, Mich. (W. F. Zabriskie.)
Gale, L. E., American Trading Co., Hankow, China.
Galien Concrete Tile Co., Galien, Mich. (C. A. Roberts.)
Gardiner, J. B. W., 50 Church St., New York, N. Y.
Gardner, Franc E., 3123 Bloomingdale Road, Chicago, Ill. (Gardner-Barada Chemical Co.)
Gardner, Franc J., 134 S. La Salle St., Chicago, Ill. (Atlas Portland Cement Co.)
Garties, George, 1100 Telephone Bldg., Cincinnati, Ohio.
Gaskill Constr. Co., 302 Planters Bank Bldg., Wilson, N. C. (W. H. Gaskill.)

GASTON, H. F., Holland, Mich.

GEDNEY Co., K. H., Hastings, Neb. (Kenneth H. Gedney.)

GEDNEY, RALPH, 1413 "F" St., N. E., Washington, D. C.

GENERAL BUILDING Co., INC., 524 Harrison Ave., Boston, Mass. (H. W. Marshall.)

GENERAL FIREPROOFING Co., Youngstown, Ohio. (W. B. Turner.)

GERMUNDSSON, TH., 1421 Maple Ave., Evanston, Ill.

GEUPEL, CARL M., c/o Thompson & Binger, Inc., 922 Hume Mansur Bldg., Indianapolis, Ind.

*GIANT PORTLAND CEMENT Co., Pennsylvania Bldg., Philadelphia, Pa. Charles F. Conn, Pres.)

GIESECKE & HARRIS, Architects, 520 Littlefield Bldg., Austin, Tex. (Munsey Wilson.)

GIL, LUIS ROBLES, 9/a Durango, Num. 159 Mexico, D. F., Mexico.

GILES, ALLEN LESTER, 147 Milk St., Boston, Mass. (Stone & Webster.)

GILES, ROY T., 218 New Bern Ave., Raleigh, N. C.

GILKEY, PROF. HERBERT J., Room 212 Engr. Bldg., University of Colorado, Boulder, Colo.

GILL, GRAYSON, 105 Roshelle Ave., Philadelphia, Pa.

GILLEWICZ, ZDZISLAW, Nowdgrodzka 25, Warsaw in Poland.

GILLIGAN, WILLIAM H., 31 Union Square, New York City. (Truscon Steel Co.)

GILLIS, W. E., Edgerton, Ohio. (Edgerton Cement Works.)

GILMAN, CHARLES, 50 Church St., New York City. (Massey Concrete Products Corp.)

GINDER, J. W., 439 Treasury Bldg., Washington, D. C.

GINSBERG-PENN Co., 18 E. 41st St., New York, N. Y. (Frank I. Ginsberg.)

GIRAUD, LEON B., Apartado 8713 "J," Mexico, D. F.

GLEASON, KATE, Commercial St., East Rochester, N. Y.

GLEASON, ROBERT W., 634 Madison Ave., Paterson, N. J.

*GLENS FALLS PORTLAND CEMENT Co., 205 Lower Warren St., Glens Falls, N. Y. (G. F. Boyle.)

*GLENS FALLS PORTLAND CEMENT Co., Glens Falls, N. Y. (G. F. Boyle, Jr.)

GEO. J. GLOVER CONSTRUCTION Co., INC., 1033 Whitney Bank Bldg., New Orleans, La.

GODFREY, EDWARD, Monongahela Bank Bldg., Pittsburgh, Pa.

GODLEY, S. S. AND G. H., 716 Southern Railway Bldg., Cincinnati, Ohio. (George H. Godley.)

GOETZ, JOHN A., Mattoon, Ill.

GOLDBECK, A. T., 515 14th St., Washington, D. C. (Bureau of Public Roads.)

GOLDIE MFG. Co., Trenton Ave. and P. R. R., Wilkinsburg, Pa. (Wm. Goldie, Jr.)

GOLDSMITH METAL LATH Co., THE, 3rd and Eggleston Ave., Cincinnati, Ohio. (Louis I. Zogoren.)

Gonnerman, H. F., 1951 W. Madison St., Chicago, Ill.
Gottschalk, L. F., Columbus, Neb.
Gould, Frank D., Fairmont, Minn.
Gould, Harley J., c/o Ferro-Concrete Constr. Co., Cincinnati, Ohio.
Gould Contracting Co., 1214 Ind. Life Bldg., Nashville, Tenn. (C. B. Wilson.)
Gow, Charles R., 957 Park Square Building, Boston, Mass.
Gram, Lewis M., 912 Oakland Ave., Ann Arbor, Mich. (University of Michigan.)
Granite Concrete Block Co., Ltd., 832 Weston Road, Toronto, Ont. (J. A. Livingston, Pres.)
Graves, Frank W., 326 Beaver Hall Hill, Montreal, Canada. (J. S. Archibald.)
Gray Concrete Co., Thomasville, N. C. (F. B. Gray.)
Gray Construction Co., Ltd., J. V., 541 Queen St., E., Toronto, Ont. (R. J. Fuller.)
Gray, Howard Allison, 862 Park Square Bldg., Boston, Mass. (Morton C. Tuttle Co.)
Great Eastern Gravel Corp., Pt. Jefferson, N. Y. (Geo. D. Perry, Treas.)
Great Western Portland Cement Co., 410 Land Bank Bldg., Kansas City, Mo. (Page Golsan.)
Green, J. Singleton, Jr., "Ravenshore," Marine Parade Hythe, Kent, England.
Green, Victor E., Industrial Research Laboratory, Gas Dept., Council House, Birmingham, England.
Greene, Roy L., Court House, Chehalis, Wash.
Greenfield, Arthur, Inc., 1 Union Sq. West, New York, N. Y. (Arthur Greenfield.)
Greenman, Russell S., State Engineer's Dept., Albany, N. Y.
Gretsch Construction Co., 50 E. 42nd St., New York City. (Herbert Gretsch.)
Grinter, Linton E., 219 Engineering Hall, Urbana, Ill. (University of Illinois.)
Grun, Richard, Direktor am Forschungsinstitut der Huttenzement-Industrie, Dusseldorf, Germany.
Guarantee Construction Co., 140 Cedar St., New York, N. Y. (Edward Burns.)
Gulf Concrete Pipe Co., Central Park Station, Houston, Texas. (N. A. Eppes.)
Hadley, H. M., 803 Seaboard Bldg., Seattle, Washington. (Dist. Engr., Portland Cement Assn.)
Hagener, Arthuur, 70 Union Bldg., Cleveland, Ohio.
Haggart, C. N., 331 4th Ave., Pittsburgh, Pa.
Hahn, Frank E., 629 Chestnut St., Philadelphia, Pa.
Hanks, Inc., Abbot A., 624 Sacramento St., San Francisco, Calif.

HALL, EDWIN C., 1037 45th St., Milwaukee, Wis.

HALL, QUINCY A., 212 Metropolitan Bank Bldg., St. Paul, Minn.

HALL CONSTRUCTION CO., 406 Board of Trade Bldg., Indianapolis, Ind. (R. T. Fatout, Secy.)

HALL & STEVENSON, 409 White Bldg., Seattle, Wash. (J. H. Stevenson.)

HAMILTON, CHARLES T., 310 London Bldg., Vancouver, B. C., Can.

HAMILTON GRAVEL CO., North Third St., Hamilton, O. (W. P. Watson, Sec'y-Treas.)

HAMMILL, HAROLD B., 42 Portsmouth Road, Piedmont, Oakland, Calif.

HAMMITT, ANDREW B., 312 Broad Street Bank Bldg., Trenton, N. J. (Par-Lock Appliers of N. J.)

HANNAFORD, H. ELDRIDGE, 1024 Dixie-Terminal Bldg., Cincinnati, Ohio. (S. Hannaford & Sons.)

HANSARD, ORREN H., Tenn. Dept. of Highways and Public Works, Nashville, Tenn.

HANSEN CO., L., 3617 E. 23rd St., Kansas City, Mo. (L. Hansen.)

HANSON, E. S., Box 498, Chicago, Ill. (Assoc. Editor, International Trade Press, Inc.)

HARDING, E. C., Ferro Concrete Construction Co., 3rd and Elm Streets, Cincinnati, Ohio.

HARDY, RICHARD, 1011 James Bldg., Chattanooga, Tenn. (Dixie Portland Cement Co.)

HARESNAPE, V., 800 Corporation Bldg., Los Angeles, Calif. (Riverside Portland Cement Co.)

HARGEN, STANLEY, 133 Rutland Road, Brooklyn, N. Y. (Standard Oil Co.)

HARIG CONSTRUCTION CO., ROBT., 2174 Western Ave., Cincinnati, Ohio. (Ben Harig.)

HARMS, H. J., Rotterdam, Holland. (Continental Petroleum Co.)

HARNISH, JOHN, 737 A. G. Bartlett Bldg., Los Angeles, Calif. (Austin Co. of California.)

HARRINGTON, JOHN LYLE, 1012 Baltimore Ave., Kansas City, Mo. (Harrington, Howard & Ash.)

HARRIS, C. P., Huron Portland Cement Co., Alpena, Mich.

HARRIS, WALLACE R., 4 Augusta St., Oak Park, Ill.

HARRISBURG BUILDING BLOCK CO., Cameron and Reilly Sts., Harrisburg, Pa. (J. Edwin Rutter.)

HARRISON CONSTRUCTION CO., J. S., 2012 Amicable Bldg., Waco, Texas. (C. H. Harrison.)

HART, R. E., 167 Eighth Ave., N., Nashville, Tenn.

HART, W. E., 111 W. Washington St., Chicago, Ill. (Portland Cement Assn.)

HATT, K. A., 1904 Washington Blvd., Maywood, Ill.

HATT, WILLIAM KENDRICK, Purdue University, Lafayette, Ind.

HAVLIK, R. F., Mooseheart, Ill.

HAWAIIAN CONTRACTING CO., 854 Kaahumanu St., Honolulu, T. H. (H. P. Benson.)

Hawkins, J. C., Waterworks Department, City Hall, Cape Town, South Africa.

Hawkins, Paul J., 1607 Merchant Bank Bldg., Indianapolis, Ind. (Crawfordsville Foundry Co.)

Hawley, John B., 403 Cotton Exchange Bldg., Ft. Worth, Texas.

Hawley, Wm. H., 2965 Madison Ave., Granite City, Ill.

Hay, Wm. Wren, Clinton Ave., R. F. D. 2, Plainfield, N. J.

Hayes, J. E., Engineering Corp., Tientsin, China.

Hayley, Harry, 171 Waller St., Ottawa, Canada.

Hayward, Harrison W., Mass. Inst. of Technology, Cambridge, Mass.

Healey, Clarence, Linde-Griffith Co., Newark, N. J.

Heather, D. S. B., Land Drainage Branch, Lands and Survey Dept., Pongakawa, Bay of Plenty, New Zealand.

Hebold, Denis O., 2401 N. Mascher St., Philadelphia, Pa.

Heidema, P. H., 322 Woodworth Ave., Glenwood, N. Y.

Heine Chimney Co., 123 W. Madison Ave., Chicago, Ill.

*Helderberg Cement Co., Albany, N. Y. (Charles R. Parks.)

Heller, Martin, Granite City, Ill.

Heller-Murray Co., 222 W. Rayen St., Youngstown, Ohio. (A. H. Heller.)

Helzer, A. E., 212 W. Washington St., Chicago, Ill.

Hench, Lynn H., 3036 "O" St., N. W., Washington, D. C.

Henderson Structural Units Co., 807 First National Bank Bldg., Pittsburgh, Pa. (Albert Henderson.)

Hendricks, Jean, 125 Quai de Valmy Paris (Xeme), France. (Poliet & Chausson.)

*Hercules Cement Corp., 1600 Walnut St., Philadelphia, Pa. (Morris King, Pres.)

Hersey Co., Ltd., Milton, 84 St. Antoine St., Montreal, Canada. (Walter C. Adams.)

Hewes, George C., 162 Lucile Ave., Atlanta, Ga.

Hewett, W. S., 2101 Harrison St., Oakland, Calif.

Heyworth, James O., Harvester Bldg., Chicago, Ill.

Hibbs, Manton E., 1423 N. 15th St., Philadelphia, Pa.

Hilker Supply Co., 16th and State Sts., Granite City, Ill. (E. W. Hilker.)

Hill, Roger F., 408 W. Fort St., Detroit, Mich.

Hillam, A. J., 2280 47th Ave., Oakland, Calif.

Hindman, W. S., 3611 Terrace St., Pittsburgh, Pa.

Hinton, Geo. B., Apartado, P. O. Box 60, Mexico, D. F., Mexico.

Hirschberg, Walter P., 218 Stephenson Bldg., Milwaukee, Wis. (Federal Engineering Co.)

Hitchcock, Frank A., Washington, D. C. (Bureau of Standards.)

Hobbs, Albert C., 537 Congress St., Portland, Me. (John P. Thomas, Architect.)

Hoeffer & Co., Chamber of Commerce Bldg., Chicago, Ill. (Alexander C. Warren.)

HOFF, J. HAAKON, 208 S. La Salle St., Chicago, Ill. (American Bridge Co.)
HOFF, OLAF, 50 Church St., New York City.
HOGAN, C. L., c/o Knickerbocker Portland Cement Co., 100 State St., Albany, N. Y.
HOLABIRD & ROCHE, 104 S. Michigan Ave., Chicago, Ill. (E. A. Renwick.)
HOLLISTER, S. C., 848 Land Title Bldg., Philadelphia, Pa.
HOLLOW BUILDING TILE ASSOCIATION, 1409 Conway Bldg., Chicago, Ill. Frank J. Huse.)
HOLLYWOOD BUILDING BLOCK Co., North Plymouth, Pa. (Henry C. Parker, Sec'y and Treas.)
HOLM, W. M., c/o Chief Engr., A. T. & S. P. Ry., Amavillo, Texas.
HOLMES, A. R., LTD., 6 Hayden St., Toronto, Canada.
HOLMES, FRANCIS, 248 Lambton Quay, Wellington, New Zealand.
HOLMGREN, ROBERT N., 104 S. Michigan Ave., Chicago, Ill. (Richard E. Schmidt, Garden & Martin.)
HOLT, H. A., Concrete Investigation Dept., 143 Grosvenor Road, London, S. W. 1, England.
HOLTZMAN, S. F., 244 Madison Ave., New York City.
HOOD, RAYMOND M., 40 W. 40th St., New York, N. Y.
HOOL, GEORGE A., College Hills, Madison, Wis. (University of Wisconsin.)
HOOVER, A. P., 150 Janvrin Ave., Bronxville, N. Y.
HOPKINS, RALPH Z., 2576 Hurlbut Ave., Detroit, Mich. (Hudson Motor Car Co.)
HORN, A. E., Hancock and Bodine Sts., Long Island City, N. Y. (A. C. Horn Co.)
HORN, H. M., 17 Battery Place, New York City.
HORNER, WESLEY W., 300 City Hall, St. Louis, Mo.
HORR, GEORGE E., 244 Madison Ave., New York City. (Turner Construction Co.)
HOUK, HOWARD H., U. S. Bureau of Public Roads, Montgomery, Ala.
HOUSEMAN ROOFING CO., INC., 1521 Pierre Ave., Shreveport, La. (G. A. Houseman, Pres.)
HOWE, C. D., The Whelan Bldg., Port Arthur, Ont., Canada.
HOWE, H. N., 76 Porter Bldg., Memphis, Tenn.
HOWES, BENJAMIN A., 70 Fifth Ave., New York City.
HOWES & MCGINTY, INC., 64 Whitman St., New Bedford, Mass. (John J. McGinty.)
HOYER-ELLEFSEN, P. O. Box 463, Kristiania, Norway. (August Gundersen.)
HOYT, W. A., Altoona, Pa.
HUDSON'S CONCRETE CO., LTD., Customs St., Auckland, New Zealand.
HUDSON, JAMES A., Memphis, Tenn. (Dist. Engr., Portland Cement Assn.)
HUDSON, R. J. H., Public Works Dept., Ranchi, India.
HUEBER BROTHERS BUILDERS, INC., 243 Baker Ave., Syracuse, N. Y. (P. J. Hueber.)
HUGGER BROS. CONSTR. CO., Montgomery, Ala.

Hughes, L. E., 315 W. 98th St., New York, N. Y.
Hughes, R. G., 152 Market St., Paterson, N. J. (John W. Ferguson Co.)
Hukmani, S. D., Sukkur Barrage Rohri, Sind, India.
Humboldt Gravel and Tile Co., Humboldt, Iowa. (C. L. Douthett, Vice-Pres.)
Hundhausen, Wm., 421 Barker Ave., Peoria, Ill.
Hume, Albert S., Cangallo 465, Buenos Aires, Argentine.
Hume Pipe Co. (South Africa), Ltd., National Bank Bldgs., Simmonds St., Johannesburg, South Africa. (Walter Wolstenholme.)
Hume, Walter Reginald, Reliance House, 301 Flinders Lane, Melbourne, Victoria, Australia. (Hume Pipe Co. of Australia, Ltd.)
Humphrey, D. S., Euclid Beach Park, Cleveland, Ohio. (The Humphrey Co.)
Humphrey, Richard L., 805 Harrison Bldg., Philadelphia, Pa.
Hunt & Co., Robert W., 53 Park Place, New York City. (J. F. Davis.)
Hunter, Geo. H., Santa Barbara, Calif.
Hurlburt, R. W., 100 Jarvis St., Toronto, Ont.
*Huron Portland Cement Co., 1525 Ford Bldg., Detroit, Mich. (John W. Boardman.)
Hustad Co., The, 126 S. 9th St., Minneapolis, Minn. (A. P. Hustad, Pres.)
Hutchinson, G. W., 11 Broadway, New York, N. Y. (Eastern Manager, Concrete Dept., Celite Products Co.)
Hutter Constr. Co., 128 Western Ave., Fond du Lac, Wis. (Geo. F. Hutter.)
Hyde, Stanley T., 212 9th St., Bremerton, Wash.
Hydro-Electric Power Comm. of Ontario, 190 University Ave., Toronto, Ont.
Ideal Cement Stone Co., Omaha, Neb. (N. J. Peterson, Pres.)
Ideal Concrete Const. Co., 455 Rowell Ave., Joliet, Ill. (Gilbert Cooper.)
Illinois Steel Co., Chicago, Ill. (T. J. Hyman.)
Illinois-Wisconsin Concrete Pipe and Tile Co., Beloit, Wis. (Chas. E. Richardson.)
Immel Construction Co., 98 N. Main St., Fond du Lac, Wic. (Harry W. Mabie, Jr.)
Independent Block & Cement Co., 2102 S. Harding St., Indianapolis, Ind. (George L. Bradshaw.)
Independent Concrete Pipe Co., 201 N. West St., Indianapolis, Ind. (Howard Schurmann.)
*Indiana Portland Cement Co., 808 Continental National Bank Bldg., 17 N. Meridian St., Indianapolis, Ind. (Marshal Beck, Treas.)
Indiana Sand and Gravel Producers' Assn., 603 Occidental Bldg., Indianapolis, Ind. (A. M. Brown, Pres.)
Industrial Engineering Co., 30 Church St., New York, N. Y. (D. Traver Miller.)

INGBERG, S. H., Bureau of Standards, Washington, D. C.
INGEMANSON, THURE W., 5944 W. Erie St., Austin Sta., Chicago, Ill.
*INLAND STEEL CO., First National Bank Bldg., Chicago, Ill. (G. H. Jones.)
INNES, R. D., c/o Aiken & Innes, Sec. 1 and 2, Welland Ship Canal, St. Catherines, Ont.
INNIS, R. L., Govt. Drainage Dept., Thornton, Bay of Plenty, New Zealand.
INSLEY, WM. H., P. O. Box 167, Indianapolis, Ind. (Insley Mfg. Co.)
*INSLEY MFG. CO., P. O. Box 167, Indianapolis, Ind. (Wm. H. Insley.)
*INSLEY MFG. CO., P. O. Box 167, Indianapolis, Ind. (Alvin C. Rasmussen.)
INTERLOCKING CEMENT STAVE SILO CO., 709 S. Wichita St., Wichita, Kans. (Kent Merry, Pres.)
*INTERNATIONAL CEMENT CORP., 342 Madison Ave., New York City.
IOWA CONCRETE PRODUCTS ASSN., 405 Hubbell Bldg., Des Moines, Iowa. (Ross Dowell.)
IRWIN, A. C., 111 W. Washington St., Chicago, Ill. (Portland Cement Assn.)
IRWIN, ORLANDO W., 1128 Ford Ave., Youngstown, Ohio. (Truscon Steel Co.)
IRWIN & LEIGHTON, 126 N. 12th St., Philadelphia, Pa. (E. M. Campbell.)
JACKSON, A. L., 7 S. Dearborn St., Chicago, Ill.
JACKSON, F. H., U. S. Bureau of Public Roads, Washington, D. C.
JACKSON, R. B., 527 W. Ganson St., Jackson, Mich.
JACKSON-LEWIS CO., LTD., THE, Ryrie Bldg., 229 Yonge St., Toronto, Ont. (C. Blake Jackson, Pres.)
JACKSONVILLE CONCRETE PRODUCTS CO., 530 Riverside Ave., Jacksonville, Fla. (Fred C. Hedrick.)
JACOBY, H. S., 6523 Euclid Ave., Cleveland, Ohio. (H. K. Ferguson Co.)
JAMESTOWN BLOCK & TILE CO., INC., P. O. Box 712, Jamestown, N. Y. (N. B. C. Stiteler, Pres.)
JEFFERS, PAUL E., 720 Pacific Finance Bldg., Los Angeles, Calif.
JEFFREY, W. MCKENZIE, Palmerston N., New Zealand. (Hume Pipe Co. (Australia), Ltd.)
JELLICK, J. E., 921 Merchants Nat'l Bank Bldg., Los Angeles, Calif. (Dist. Engr., Portland Cement Assn.)
JENRICK, WM. F., 147 Milk St., Boston, Mass.
JEWETT, F. C., 16 Ontario St., S., St. Catherines, Ont.
JEWETT, JOHN Y., Administration Bldg., Balboa Park, San Diego, Calif.
JEWKES & SONS CO., JOSEPH, 676 Montgomery St., Jersey City, N. J. (Francis R. Jewkes.)
JOGLSKAR, H. V., P. O. Bhira, District Kolaba, Bombay, Presidency, India. (Tata Construction Co., Ltd.)
JOHNSON, ALGOT F., 809 1st National Soo Line Bldg., Minneapolis, Minn.
JOHNSON CO., C. S., 1002 N. Market St., Champaign, Ill. (Chas. S. Johnson.)
JOHNSON, FRANK H., Ellsworth, Wis.

JOHNSON, LEWIS J., Harvard University, Cambridge, Mass.
JOHNSON, N. C., 342 Madison Ave., New York City.
JOHNSON, T. H., 319 Iowa Bldg., Sioux City, Iowa.
JOHNSON, VIRGIL L., 19th and Chestnut Sts., 801 Denckla Bldg., Philadelphia, Pa.
JOHNSTON, ROBERT S., 3604 McKinley St., N. W., Washington, D. C. (Bureau of Standards.)
JOHNSTON, P. H., 1010 Graham Bldg., Jacksonville, Fla. (Dist. Engr., Portland Cement Assn.)
JONES, BEVAN, 342 Madison Ave., New York City.
JONES, D. W., Supt. of Buildings City Hall, Binghamton, N. Y.
JONES CONSTRUCTION CO., H. N., Alamo Theater Bldg., San Antonio, Texas. (C. M. Bushick, Vice-Pres.)
JONES, WILLIAM M., 152 Market St., Paterson, N. J.
KAHN, ALBERT, Marquette Bldg., Detroit, Mich.
KAHN, GUSTAVE, Youngstown, Ohio. (Truscon Steel Co.)
KAISER, B. J., 801 Keystone Bldg., Pittsburgh, Pa.
KALMAN FLOOR CO., 410 N. Michigan Ave., Chicago, Ill. (C. E. Cooke.)
*KALMAN STEEL CO., 410 Michigan Ave., Chicago, Ill. (William S. Thomson.)
*KALMAN STEEL CO., 410 N. Michigan Ave., Chicago, Ill. (A. P. Clark.)
*KANSAS PORTLAND CEMENT CO., Federal Reserve Bank Bldg., Kansas City, Mo. (J. A. Lehaney, Vice-Pres.)
KAPADIA, B. F., Abdulla Bldgs., No. 2, Tram Terminus Parel, Bombay, India.
KAPP, P. B., 707 W. College Ave., State College, Pa. (Penn. State College.)
KATTELLE, WALTER R., Western Electric Co., 110 William St., New York City.
KAUFMAN, DAVID M., 30 Church St., Room 573 E, New York, N. Y.
KAUFMAN, FOREST, 911 Gloyd Bldg., Kansas City, Mo. (Dist. Mgr., Portland Cement Assn.)
KAYE, LISTER L., Southern Works, Rugby, England.
KELLEY, FREDERICK W., 126 State St., Albany, N. Y. (Helderberg Cement Co.)
KELSO, JAMES A., Industrial Laboratories, Edmonton, Can. (University of Alberta.)
KELTY, EMER G., 122 N. 51st St., Philadelphia, Pa. (Consolidated Expanded Metal Co.)
KEMMER, A. E., Lafayette, Ind.
KENT, COL., H. VAUGHAN, 34 Victoria St., London, S. W. I, England.
KENT, COL., H. VAUGHAN, 34 Victoria St., London, S. W. 1, England.
KENWORTHEY, EDW. M., 2311 W. 11th St., Wilmington, Del.
KERR, LINTON, 147 Milk St., Boston, Mass.
KESLINGER, ALBERT, Box 99, Oswego, Ill.

KIENSTRA BROS. FUEL AND SUPPLY CO., Wood River, Ill. (Frank T. Kienstra.)

KIKUCHI, AITRO, TOA CEMENT CO., LTD., Amagasaki, near Osaka, Japan.

KINDLE, GEORGE C., Pitman, N. J.

KING, A. W., 410 N. Michigan Ave., Chicago, Ill. (Kalman Floor Co.)

KINGSBURY, C. T., 216 Woodward Bldg., Washington, D. C. (Rosslyn Steel and Cement Co.)

KINNEY, WILLIAM M., 111 W. Washington St., Chicago, Ill. (Portland Cement Assn.)

KIRK, KARL Q., 520 McCalie Ave., Chattanooga, Tenn.

KITCHEN, R. R. & CO., 802 National Bank Bldg., Wheeling, W. Va. (R. R. Kitchen.)

KITTANNING SALES CO., INC., 45 E. 17th St., New York, N. Y. (Robert Recker, Sec'y.)

KLAGSHAMNS CEMENTVERKS AKTIEBOLAG, Postfack 1068, Stockholm, Sweden. (Ivar Olsson.)

KLEIN, W. H., Richard City, Tenn. (Dixie Portland Cement Co.)

KLEINLOGEL, PROF. ADOLF, Roquetteweg 33, Darmstadt, Germany. (Hessen.)

KLEPACH CONSTRUCTION CO., Cedar Rapids, Iowa. (John Klepach.)

KLEYBOLTS, RUDOLPH, 515 Union Central Bldg., Cincinnati, Ohio. (Prudential Builders, Inc.)

KLINGBERG, W. EARL, 318 Main St., Springfield, Mass.

KLINGER, W. A., Warnock Bldg., Sioux City, Iowa.

KLOCK, MORGAN B., 189 Park Ave., Rochester, N. Y.

KNOPH, OLAF, Prinsensgt 26b, Elevator, Kristiania, Norway.

KNOWLTON, WINFIELD B., 69 Salem St., Andover, Mass.

KOBER, WM., CO., c/o Adensite Co., Inc., 116 W. 39th St., New York City.

*KOEHRING COMPANY, 31st and Concordia Ave., Milwaukee, Wis. (E. H. Lichtenburg.)

*KOEHRING COMPANY, 4940 N. 8th St., Philadelphia, Pa. (P. Koehring.)

KOERNER & CO., C. A., 318 E. Burnett, Louisville, Ky. (R. J. Sweeney.)

KOERNER, CARL A., 908 Syndicate Trust Bldg., St. Louis, Mo.

KOHCHI, M., Onoda Cement Co., Yamaguchi-Ken, Japan.

KOLB, F. X., 1225 E. Grand Blvd., Detroit, Mich.

KOMURA, MANGORO, Yotsu Kuracho, Fukushi, Maken, Japan. (Iwaki Cement Co., Ltd.)

KONSTANT, NICKALAS Z., 1473 Carmen Ave., Chicago, Ill.

KOPITKE, O. F., Wabash and 15th Sts., Toledo, Ohio. (The Cettins Kopitke Co.)

*KOSMOS PORTLAND CEMENT CO., 614 Paul Revere Bldg., Louisville, Ky. (O. N. Clarke.)

KRAUSE, G. E., Juneau, Alaska.

KRAUSE, L. B., 231 S. La Salle St., Chicago, Ill.

KRAUSE, MARK C., 120 W. 4th St., Williamsport, Pa.

KREBS COMPANY, A. J., Walton Bldg., Atlanta, Ga. (A. J. Krebs.)

Krecker, Raymond H., c/o Phila. & Reading Ry., 9th and Spring Garden Sts., Philadelphia, Pa.

Kressly, Paul E., 732 H. W. Hellman Bldg., Los Angeles, Calif.

Krier, George H., 814 E. 94th St., Brooklyn, N. Y.

Kuhn, Percy C., c/o Wogan & Bernard, 1002 Title Guarantee Bldg., New Orleans, La.

Kuo, Tienpang, 2 S. Clinton St., Trenton, N. J.

Kvitrud, I., 754 Builders Exchange, Minneapolis, Minn.

Lackey, Henry W., 2036 E. 84th St., Chicago, Ill.

Laclede Steel Co., 1317 Arcade Bldg., St. Louis, Mo. (W. L. Allen.)

Lagaard, M. B., Experimental Engineering Bldg., Minneapolis, Minn. (University of Minnesota.)

Lakdavala, Burjor M., c/o H. H. Daruwala, Esq., Outfort, Broach, Bombay, India.

Lake, Simon, Milford, Conn.

Lakewood Engineering Co., Cleveland, Ohio. (Lion Gardner, Vice-Pres.)

Lamb Co., Robert E., 843 N. 19th St., Philadelphia, Pa. (Robert E. Lamb.)

Lambert, Walter E., 2028 Lincoln St., Evanston, Ill.

Lambie, J. Edward, 5901-5999 Hydraulic Ave., Cleveland, Ohio. (Lambie Concrete House Corp.)

Lambie, Joseph S., Parkman Blvd., Pittsburgh, Pa. (University of Pittsburgh.)

Lancaster, Lionel W., 8 McLaren St., Red Bank, N. J.

Lancaster Concrete Tile Co., Lancaster, Pa. (Henry Boettcher.)

Lander, R. S., Burwell Bldg., Knoxville, Tenn. (Shearman Concrete Pipe Co.)

Landor, Edward J., 634 Renekert Bldg., Canton, Ohio.

Lane, H. A., Baltimore and Ohio Central Bldg., Baltimore, Md. (Baltimore & Ohio Railroad Co.)

Lang, Philip George, Jr., 300 Baltimore & Ohio Bldg., Baltimore, Md. (Engr. of Bridges.)

Lapham, John R., 1829 G St., N. W., Washington, D. C. (George Washington University.)

Larkin, Edward C., No. 10 Florence Apts., Warren St., Dayton, Ohio.

Larson, Reuben Lawrence, 4-6 Yuen Ming Yuen Road, Shanghai, China. (Anderson, Meyer & Co., Ltd.)

"La Tolteca," Cia de Cemento, Portland, S. A., Independencia 8, P. O. Box 233, Mexico, D. F. Mexico. (G. H. E. Vivian.)

Lavelle, J., General Assurance Bldg., Bay and Temperance Sts., Toronto, Ont. (Alfred Rogers, Ltd.)

Lavigne, Ernest T., 30 Belvidere Road, Quebec, Canada. (Quebec Provincial Dept. of Public Works & Labor.)

Lazier, F. S., Welland Ship Canal, Thorold, Ont., Canada.

*Lawrence Portland Cement Co., 302 Broadway, New York City. (J. S. Van Middlesworth.)

LEA, WILLIAM S., 809 New Birks Bldg., Phillips Square, Montreal, Que. (R. S. & W. S. Lea.)

LEACH, FRED M., 798 Detroit Savings Bank Bldg., Detroit, Mich.

LEAVER, R. J., 49 Swan St., Lawrence, Mass.

LEE, KUNG, c/o Building Sect., Yuen Ming, Yuen Rd., Shanghai, China. (Anderson, Meyer & Co., Ltd.)

LEE, W. HAMILTON, South Plainfield, N. J.

LEEDS & BARNARD, 705 Central Bldg., Los Angeles, Calif. (Chas. T. Leeds.)

LEFFLER, RALPH R., 7021 Oriole Ave., Chicago, Ill.

*LEHIGH PORTLAND CEMENT CO., Allentown, Pa. (H. M. Trexler, Pres.)

*LEHIGH PORTLAND CEMENT CO., Allentown, Pa. (H. M. Trexler, Pres.)

LEHIGH PORTLAND CEMENT CO., Young Bldg., Allentown, Pa.

LENGST, G. J., 212 W. Bluff St., Prairie du Chien, Wis. (Prairie Concrete Prod. Co.)

LENT, RICHARD P., Box 268, Wilson, N. C.

LEONARD, JOHN B., 57 Post St., San Francisco, Calif.

LEONARD, W. H., 800 Corporation Bldg., Los Angeles, Calif. (Riverside Portland Cement Co.)

LESLEY, ROBERT W., 611 Pennsylvania Bldg., Philadelphia, Pa.

LEUNARTZ, ALLAN E., 10 Farleigh St., Ashfield, Sydney, Australia.

LEVISON, ARTHUR A., Chief Engr., Road Dept., Pittsburgh, Pa. (Blaw-Knox Co.)

LEWIS, GEO. H., Malden, W. Va.

LIBBERTON, J. H., 40 Rector St., New York City. (General Chemical Co.)

LIEBERMAN & HEIN, 190 N. State St., Chicago, Ill. (E. Lieberman.)

LILLEY CONCRETE PRODUCTS CO., Aurora, Ill. (L. W. Lilley.)

LIND, PETER & CO., 2 Central Bldg., Westminster, London, S. W. 1, England.

LINDAU, A. E., 10 S. La Salle St., Chicago, Ill. (American System of Reinforcing.)

LINDSAY & CO., W W., 902 Harrison Bldg., Philadelphia, Pa. (James C. Newlin, Vice-Pres.)

LINDSLEY CO., C. E., 888 Clinton Ave., Irvington, N. J. (C. E. Lindsley.)

LINSTRUM, A. C., 607 Hubbell Bldg., Des Moines, Iowa.

LINDSTROM, ROBERT S., 203 S. Dearborn St., Chicago, Ill. (Advance Waterproof Cement Co.)

LIPSCOMB, P. T., Crockett, Texas.

LITTER, F. J., 8 W. 40th St., New York City. (The Frederick Snare Corp.)

LIVERMORE, A. C., Mgr. Westinghouse Air Brake Home Bldg. Co., Wilmerding, Pa.

LIVERMORE, JOSEPH D., Route 6, Madison, Wis.

LOCK JOINT PIPE CO., P. O. Box 21, Ampere, N. J. (J. E. Longley.)

LOCK JOINT PIPE CO., Ampere, N. J. (F. F. Longley.)

*LOCK JOINT PIPE CO., Ampere, N. J. (A. M. Hirsh, Pres.)

Locke, Clyde E., 905 Ellicott Square, Buffalo, N. Y. (A. E. Baxter Eng. Co.)

Lockhardt, William F., 347 Madison Ave., New York, N. Y.

Lockwood, Greene & Co., 24 Federal St., Boston, Mass. (Library.)

Loeb, Henry, II, c/o Loeb Stone Company, Memphis, Tenn.

Loeber, Charles, P. O. Box 1612, Richmond, Va.

Loehler, Paul F., 1300 Kahnia St., N. W., Washington, D. C., Tacoma Station.

Logeman, R. T., 208 S. La Salle St., Chicago, Ill. (American Bridge Co.)

Loney, Neil M., 250 Park Ave., New York, N. Y. (Thompson-Starrett Co.)

Long & Thorshov, 1028 Andrus Bldg., Minneapolis, Minn.

Loomis & Sons Co., C. H., 306 Jelliff Ave., Newark, N. J. (C. H. Loomis, Pres.)

Lord, Arthur R., 140 S. Dearborn St., Chicago, Ill.

Lorentzen, G. F., Cementfabrik Nerge CE-NO Portland Cement A/s., Oslo, Norway.

Lorenz Co., P. H., 413 Peoples Bank Bldg., Moline, Ill. (F. R. Dewend.)

Lorenz, Victor S., 129 Wadsworth Ave., Apt. 61, New York, N. Y.

Loring, Louis T. C., 10 High St., Boston, Mass. (Dist. Engr., Portland Cement Assn.)

Los Angeles Concrete Tile Co., 432 I. W. Hellman Bldg., Los Angeles, Calif. (Harry Soderberg.)

Los Angeles Harbor Dept., Berth 90, San Pedro, Calif. (Geo. F. Nicholson.)

Lothian, Albert J., 230 Chatham St., W., Windsor, Ont., Can.

*Louisville Cement Co., 315 Guthrie St., Louisville, Ky. (W. S. Speed, Pres.)

Love, Harry J., 933 Leader-News Bldg., Cleveland, Ohio. (Nat. Slag Assn.)

Lowell, John W., 208 S. La Salle St., Chicago, Ill. (Universal Portland Cement Co.)

Lubin, Frank, 11 Goodell St., Buffalo, N. Y. (Turner Construction Co.)

Luety, George, 1405 Prairie Ave., Beloit, Wis.

Lundoff-Bicknell Co., The, 5716 Euclid Ave., Cleveland, Ohio. (C. W. Lundoff.)

Luten, Daniel B., 1056 Lemcke Annex, Indianapolis, Ind. (Luten Engr. Co.)

Luzerne County Road and Bridge Dept., Wilkes-Barre, Pa. (Robert L. Williams.))

Lynam, Major C. G., R. E., Public Works Dept., Bagdad, Mesopotamia.

Lynch, W. J., 104 S. Michigan Ave., Chicago, Ill. (Thompson-Starrett & Co.)

MacBeth, Norman, 800 Corporation Bldg., Los Angeles, Calif. (Riverside Portland Cement Co.)

Macgowan, Ernest S., 836 Security Bldg., Minneapolis, Minn.

McBurney, J. W., c/o Standard Paint & Lead Works, Cleveland, Ohio.

McCARTHY, P. A., Box 794, Lufkin, Texas. (Commercial and Industrial Engrg. Co.)

McCARTHY, T. V., Box 245, Niagara Falls, Ont., Canada.

*McCLATCHY, JOHN H., 848 Land Title Bldg., Philadelphia, Pa.

McCLELLAN & JUNKERSFELD, 68 Trinity Pl., New York City. (P. Junkersfeld.)

McCRADY, LOUIS DE B., c/o Canadian Explosives, Ltd., 120 St. James St., Montreal, P. Q., Canada.

McCULLOUGH, F. M., Carnegie Institute of Technology, Pittsburgh, Pa.

McDANIEL, ALLEN B., Investment Bldg., Room 712, Washington, D. C.

MACDONALD, EARLE, Riverside, Calif. (Riverside Portland Cement Co.)

McEWEN, A. B. c/o Wm. I. Bishop, Ltd., 822 New Birks Bldg., Montreal, P. Q., Canada.

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McINTYRE, WILLIAM A., 809 Flanders Bldg., Philadelphia, Pa. (Atlas Portland Cement Co.)

McINTYRE MACHINERY CO., 708 Empire Bldg., Detroit, Mich. (A. E. Carpenter, Secy.)

McKINSTRY, ROSS W., 205 Kenmore Ave., Elmhurst, Ill.

McLACHLAN, R. C., c/o J. P. Porter, Standifer & Porter Bros., St. Catherines, Ont.

McLEAN CONTRACTING CO., 1415 Fidelity Bldg., Baltimore, Md. (Oscar B. Coblentz, Pres.)

McLEAN, WILLIAM K., 8 Spring St., Sydney, New South Wales, Australia.

McLEOD, WILLIAM, Balgownie Ave., Gonville, Wanganui, New Zealand.

McMILLAN, E. C., 107 Clifford St., Detroit, Mich. (Kalman Floor Co.)

McMILLAN, FRANKLIN R., 1951 W. Madison St., Chicago, Ill. (Structural Materials Research Laboratory.)

McRAE STEEL CO., 16 McGraw Bldg., Detroit, Mich. (William Corman.)

McWILLIAM, R. J., London Bank Chambers, Creek St., Brisbane, Australia.

MACATEE, W. L., & SONS, Austin and Commerce Sts., Houston, Texas.

MACHNER, H. FRANK, Itabayanna, Estado Parahyba, d. N., Brazil.

MACK, THOMAS, Peoples Gas Bldg., Chicago, Ill. (Rezilite Mfg. Co.)

MACONI, G. V., 67 Church St., New Haven, Conn. (The Dwight Building Co.)

MAGUIRE CO., CHARLES E., 507 Grosvenor Bldg., Providence, R. I. (Charles A. Maguire.)

MAIN, CHARLES T., 200 Devonshire St., Boston, Mass.

MAKI, DR. H., Public Works Bureau, Home Dept. of Japan, Tokyo, Japan.

MALMED, A. T., 1713 Sansom St., Philadelphia, Pa. (A. T. Malmed Co.)

MALONE, JOHN A., Lancaster, Pa. (Malone & Sons.)

MALONEY, ROWLAND, Hyderabad, Deccan, India. (The Reliance Tile Works.)

MALTBY, JOHN W., Ross Bldg., Lafayette, Ind.

Maney, G. A., 3628 18th Ave., S., Minneapolis, Minn.
Manitoba, University of, Sherbrooke and Portage Sts., Winnipeg, Man. (J. N. Finlayson.)
Mantica, Albert J., 301 Journal Bldg., Albany, N. Y.
Marani, Virgil G., 844 Rush St., Chicago, Ill. (Gypsum Industries.)
Marble, William O., 508 London Bldg., Vancouver, B. C. (Hodgson, King & Marble.))
Mariscal, Frederico E., 9a Colima 292, Mexico City, Mexico.
Markland, M. B., Guarantee Trust Bldg., Atlantic City, N. J.
Marlboro Cement Co., Edmonton, Alberta, Can. (A. W. G. Clark.)
Marquess, C. H., c/o Armour & Co., Union Stock Yards, Chicago, Ill.
Marquette Cement Mfg. Co., LaSalle, Ind. (C. M. Butler.)
*Marquette Cement Mfg. Co., Marquette Bldg., Chicago, Ill. (R. B. Dickinson.)
Marsch, Louis, Morrisonville, Ill.
Marsh-Murdock Co., The, Melish and Stanton Aves., Cincinnati, Ohio. (George J. Marsh.)
Marshall, John, 528 Collins St., Melbourne, Victoria, Australia. (The Marshall Concrete Co., Ltd.)
Marshall, Jos. M., Jr., 1319 Hurt Bldg., Atlanta, Ga. (Dist. Engr., Portland Cement Assn.)
Marshall, Thos. W., 729 15th St., N. W., Washington, D. C.
Marson, John E., Aurora, Ill. (Barber-Greene Co.)
Martin, Edgar, Chicago Beach Hotel, Chicago, Ill.
Martin, Evan S., 16 Saulter St., Toronto, Ont. (James A. Wickett, Ltd.)
Martin, Frank J., 544 5th St., Port Arthur, Texas. (c/o Jefferson Constr. Co.)
Martin, G. G., Box 432, Conneaut, Ohio. (Bessemer and Lake Erie Railroad Co.)
Mason, Frank H., 2 Greylock Rd., Waterville, Me. (Central Maine Power Co.)
Massey Concrete Products Corp., Peoples Gas Bldg., Chicago, Ill. (Paul Kircher.)
Master Builders Co., The, 1836 Euclid Ave., Cleveland, Ohio. (S. W. Flesheim.)
Mauro, Francesco, First Nat. Bank Bldg., Birmingham, Ala.
Mayers, H. Winfield, No. 8 Wilson Ave., Watertown, Mass.
Maynard, Arthur J., Mass. State Farm, State Farm, Mass.
Maynicke & Franke, 25 Madison Square, North, New York City, N. Y.
Mazur, Isador, 3704 N. Penn St., Apt. 6, Indianapolis, Ind.
Mead, C. A., 165 Wildwood Ave., Upper Montclair, N. J.
Meade, Suydam Co., 342 6th Ave., Newark, N. J. (F. J. Meade.)
Meade, P. F., Denver, Colo. (Dist. Engr., Portland Cement Assn.)
Medford Concrete Co., Medford, N. J. (Harry L. King, Jr., Pres.)
Melin, O. W., Structural Engr., Western Electric Co., Hawthorne Station, Chicago, Ill.

Merchant, Archie W., 728 Hospital Trust Bldg., Providence, R. I.
Merlo, Merlo & Ray, Ltd., Ford, Ont., Canada. (Louis Alvin Merlo.)
Merriken, C. W., Tacoma Bldg., Chicago, Ill. (Gardiner & Lewis, Inc.)
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Messey, Laurel, Commerce Bldg., Ash and George Sts., Sydney, Australia.
Metcalf & Eddy, 14 Beacon St., Boston, Mass. (Frank A. Marston.)
Metzger-Richardson Company, 503 May Bldg., 529 Liberty Ave., Pittsburgh, Pa. (F. L. Metzger.)
Meyer, C. Louis, 608 Omaha National Bank Bldg., Omaha, Neb. (Concrete Engrg. Co.)
Meyer, Morrison & Co., 39 Cortlandt St., New York City. (B. A. Meyer.)
Michigan University Library, Ann Arbor, Mich.
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Miesenhelder, P. D., 640 Middle Drive, Woodruff Pl., Indianapolis, Ind. (Indiana State Highway Commission.)
Milburn Lime & Cement Co., Ltd., 59 Crawford St., Dunedin, New Zealand. (J. H. Stewart, Gen. Mgr.)
Miller, Charles R., 556 Susette St., Memphis, Tenn.
Miller, Daniel J., Bangor, Pa.
Miller & Sons' Co., H., 2565 5th Ave., Pittsburgh, Pa. (A. G. Miller.)
Miller, L. C., 610 Merchants Bank Bldg., Indianapolis, Ind. (Dist. Engr., Portland Cement Assn.)
Miller, O. L., & Co., 401 W. 17th St., Indianapolis, Ind. (A. C. Miller.)
Miller, R. M., 130 N. E. 24th St., Miami, Fla.
Miner, Joshua L., 814 Second Place, Plainfield, N. J.
Minn. Cement Construction Co., 433 Lumber Exchange, Minneapolis, Minn. (Andrew Nordloef, Mgr.)
Minshall, R. E., 242 S. Gill St., State College, Pa.
Minwax Co., Inc., 18 E. 41st St., New York, N. Y. (Albert R. Harrison, Vice-Pres.)
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Mitchell, Nolan D., 134 Beach St., South, Clarendon, Va. (U. S. Bureau of Standards.)
Modern Construction Co., Grand Junction, Iowa. (O. B. Lofstedt, Secy.)
Moeser, Victor L., Ferro Concrete Construction Co., Cincinnati, Ohio.
Mohler, John D., 2740 Duncan St., St. Joseph, Mo.
Mohr, H. A., 1007 Royster Bldg., Norfolk, Va. (Dist. Mgr. Raymond Concrete Pile Co.)
Mole, Harry H., City Engineer, Kearney, Neb.
Moline Cast Stone Co., 110 18th St., Moline, Ill. (Richard Bjoindakl.)
Mollenkof, J. P., c/o John H. McClatchy, Erdenheim, Pa.
Monarch Engineering Co., Chamber of Commerce Bldg., Buffalo, N. Y. (H. R. Wait, Pres.)
Monks & Johnson, 99 Chauncey St., Boston, Mass. (John J. Harty.)

MONOLITHIC HOLLOW CONCRETE FORM CORP., 326-330 Pacific Finance Bldg., Los Angeles, Calif. (L. J. Desenberg.)

MONTZ, A. S., 205 Strand Bldg., New Orleans, La.

MOORE, O. L., 1532-210 S. LaSalle St., Chicago, Ill.

MOORE, THOMAS, The Adensite Co., 116 W. 39th St., New York City.

MOORES-CONEY CO., 111 E. 4th St., Cincinnati, Ohio. (W. W. Coney.)

MOORE, CHAS. C., Room 305, Education Hall, University of Washington, Seattle, Wash.

MORRILL, F. W., Ferro Concrete Construction Co., 3rd and Elm Sts., Cincinnati, Ohio.

MORRIS, CLYDE T., Ohio State University, Columbus, Ohio.

MORRIS, L. E., Valley Center, Kan.

MORRISON, R. L., Assoc. Prof. Highway Engineers, University of Michigan, Ann Arbor, Mich.

MORROW, DAVID W., 4500 Euclid Ave., Cleveland, Ohio.

MORSSEN, C. M., 37 Belmont St., Montreal, Que.

MOSES, FREDERICK W., 10 Weybossett St., Providence, R. I. (Fireman Insurance Co.)

MOTO, CANDELARIO CALOR, Bureau of Municipal Works, Dept. of Interior, San Juan, P. R.

MOULTON, A. G., 812 Keefer Bldg., Montreal, P. Q., Canada. (Thompson-Starrett Co., Ltd.)

MOYER, ALBERT, 350 Madison Ave., New York City. (Vulcanite Portland Cement Co.))

MUELLER, HAROLD P., 204 Oak Lane Trust Bldg., Broad and 67th Aves., Philadelphia, Pa.

MUELLER, J. W., Palladium Bldg., Richmond, Ind.

MUIRHEAD CONSTRUCTION CO., WM., Durham, N. C. (Wm. Muirhead.)

MUNN, P. J., 147 Milk St., Boston, Mass.

MUNTZ, E. P., 403 Lehigh Valley Terminal, Buffalo, N. Y.

MURPHY, J. C., 714 Louisville Trust Bldg., Louisville, Ky.

MYLCHCREEST, GEO. LEWIS, 238 Palm St., Hartford, Conn.

MYLREA, T. D., 207 Engineering Hall, Urbana, Ill. (University of Ill.)

NAGAYA, S., Chief Engineer, Japanese Government Railway, Tokyo, Japan.

NAITO, TACHU, University of Waseda, Tokyo, Japan. (Engineering College.)

NASH, G. C., Fairport, N. Y.

*NASSAU SAND AND GRAVEL CO., 949 Broadway, New York City. (W. J. Timberman.)

NASU, AKIYA, Kawasaki Works, Nakashibuya, No. 715, Tokyo, Japan.

NATIONAL CONCRETE CONSTRUCTION CO., 54 Bd. of Trade, Louisville, Ky. (J. B. Ohligschlager.)

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NATIONAL LIME ASSOCIATION, 918 G St., N. W., Washington, D. C. (W. A. Freret.)

NATIONAL LIME ASSOCIATION, Eastern Division, 41 Bonair St., Somerville, Mass. (John J. Hurley.)

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NOICE, BLAINE, 1326 Washington Bldg., Los Angeles, Calif.

NOONAN, W. H., 334 Roy Bldg., Halifax, Nova Scotia.

NORRIS, W. H., 5131 Cypress St., Pittsburgh, Pa. (Duquesne Constr. Co.)

NORTHERN CONSTRUCTION CO., LTD., Vancouver Block, Vancouver, B. C., Canada. (Wm. Smaill, Chief Engr.)

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NORTHWESTERN STATES PORTLAND CEMENT CO., Mason City, Iowa. (G. C. Blackmore.)

NORTHWESTERN UNIVERSITY, 316 Huntington Ave., Boston, Mass. (Henry B. Alvord.)

NOVELLA, GUSTAVO, Avenida del Hipodromo, Gautemala, Guatemala, C. A.

OAKEY, CHARLES W., 412 W. Washington Ave., Madison, Wis.

O'CONNELL, N. B., 11 Goodell St., Buffalo, N. Y. (Turner Constr. Co.)

O'CONNELL, SIMON T., 237 Darragh St., Pittsburgh, Pa.

OEHMANN, JOHN W., Room 110, District Bldg., Washington, D. C. (Inspector of Buildings, D. C.)

OEHRLE, WILLIAM, 342 Madison Ave., New York, N. Y.

OESTERBLOM, I., The Truscon Steel Co., St. Helen's Court, Ballard Estate, Bombay, India.

OGDEN PORTLAND CEMENT CO., Room 521, Eccles Bldg., Ogden, Utah. (R. C. Briscoe.)

OGDEN, WILLIAM, Madison, Ind. (Rep., Lakewood Engineering Co.)

OKUBO, TOSHIYUKI (Truscon Steel Co. of Japan), Yurakucho Kojimachi, Tokyo, Japan.

OLSON, OLE K., 833 Pendido St., New Orleans, La.

Ord, William, 210 N. Clinton St., Chicago, Ill.
Ornitz, Edw. M., 5734 Wilkins Ave., Pittsburgh, Pa.
Orr, John B., 6th St., Miami, Fla.
Osborn Engineering Co., The, 7016 Euclid Ave., Cleveland, Ohio. (Bernard L. Green.)
Osborne, Raymond G., Basement, Marsh-Strong Bldg., 9th and Main Sts., Los Angeles, Calif.
Oscar, L. C., Bureau of Standards, Washington, D. C.
O'Shea, D. W., c/o H. S. Ferguson, Consulting Engr., 200 5th Ave., New York City.
*Ottawa Silica Co., Ottawa, Ill. (P. S. McDougall, Gen. Mgr.)
Overpeck, O. E., Swift & Co., Chicago, Ill.
Pacific Stone Co., 4257-8 N. W., Seattle, Wash. (A. W. Swartz, Mgr.)
Palmer Concrete Products Co., 171 Lowell St., Peabody, Mass. (Osborn Palmer, Gen. Mgr.)
Panzer, R. R., 609 Southern Ohio Bank Bldg., Cincinnati, Ohio.
Paradies, George R., 343 Madison Ave., New York, N. Y.
Parish, W. L., 1317 E. Lombard St., Davenport, Iowa.
Parker, Frank S., 280 Madison Ave., New York City. (Parker & Shaffer.)
Parrish, Deane M., P. O. Box 1223, Richmond, Va. (Economy Concrete Co. of Va., Inc.)
Parry, Charles, 316 E. Glenside Ave., Glenside, Pa.
Patel, N. T., Mashland-Price & Co., Nesbit Road, Mazagaon, Bombay, India.
Paterno, Jr., Maximino, 917 R. Hidaljo, Manila, P. R.
Pattillo, James N., 1710 Virginia Rd., Los Angeles, Calif.
Patzig, Monroe L., 206 11th St., Des Moines, Iowa.
Peabody, Dean, Jr., Room 301, Mass. Institute of Technology, Cambridge, Mass.
Pearse, Langdon, 910 S. Michigan Ave., Chicago, Ill. (Sanitary District of Chicago.)
Pearson, J. C., Young Bldg., Allentown, Pa. (Lehigh Portland Cement Co.)
Pease, B. S., 208 S. LaSalle St., Chicago, Ill. (Am. Steel & Wire Co.)
Peden, L. T., P. O. Box 341, Houston, Texas.
Peeless Artificial Stone, Ltd., 514 Coxwell Ave., Toronto, Ont., Canada. (F. C. Bee.)
Peerless Concrete Products Co., 4956 8th Ave., S., Seattle, Wash.
*Peerless Portland Cement Co., Union City, Mich. (Wm. M. Hatch.)
*Peninsular Portland Cement Co., Cement City, Mich.
*Penn-Allen Cement Co., Allentown, Pa.
*Penn-Allen Cement Co., Widener Bldg., Allentown, Pa. (W. E. Eidel.)
Penn Building Block Co., Inc., 923 Sansom St., Philadelphia, Pa. (Raymond M. Weeks.)
*Pennsylvania Cement Co., 131 E. 16th St., New York City. (William Beach.)

*PENNSYLVANIA CEMENT CO., 131 E. 16th St., New York, N. Y. (W. N. Beach, Pres.)

PENNSYLVANIA STATE HIGHWAY DEPT., Harrisburg, Pa. (W. H. Connell, Asst. State Highway Comm.)

PERKINS, RUPERT G., 315 W. 98th St., New York, N. Y.

PERMANENT MATERIALS CO., 3026 E. 1st St., Duluth, Minn. (S. B. Shepard.)

PERROT, EMILE G., Boyertown Bldg., 1211 Arch St., Philadelphia, Pa.

PERROTT, LESLIE M., Architect, 243 Collins St., Melbourne, Victoria, Australia.

PERSON, WM., Ashland, Ky. (Person Slagtex Co.)

PERRY, BRIAN R., 404 New Birks Bldg., Montreal, Canada. (MacKinnon Steel Co., Limited.)

PERRY, J. P. H., Vice-President, Turner Construction Co., 244 Madison Ave., New York City.

PERRY, L. A., Longview, Wash. (The Longview Co.)

PEYTON, LACY, Benton, Ill. (Peyton's Concrete Works.)

*PHOENIX PORTLAND CEMENT CO., Age-Herald Bldg., Birmingham, Ala. (R. E. Roscoe, Chief Chemist.)

PICKLES, WILLIAM W., 823 Bankers Trust Bldg., Philadelphia, Pa

PIERCE TESTING LABORATORIES, THE, 730-34 19th St., Denver, Col. (George Pierce, Mgr.)

PIERSON, CHARLES V., 202 S. 21st Ave., West, Duluth, Minn. (Duluth Bldrs. Supply Co.)

PIERSON CONCRETE PRODUCTS CO., 89 Dodd St., East Orange, N. J. (James T. Pierson, Vice-Pres.)

PIGOTT, JOSEPH M., Pigott-Healy Construction Co., Hamilton, Ont., Can.

PINNELL, JAS., 140 Cedar St., New York, N. Y. (Raymond Concrete Pile Co.)

PITTSBURGH TESTING LABORATORY, 7th and Bedford Ave., Pittsburgh, Pa. (A. R. Ellis, Gen'l Mgr.)

PLAGWIT, ERIC, Room 312, 311 Ross St., Pittsburgh, Pa.

PLANO CONCRETE WORKS, Box B, Plano, Ill. (Chas. A. Steward.)

PLASTIC PRODUCTS CO., 123-125 Reservoir Ave., Milwaukee, Wis. (Ralph W. Albrecht, Pres.)

PLUMER, H. E., 22 Ellicott Square, Buffalo, N. Y.

POLARIS CONCRETE PROD. CO., Box 86, W. Duluth, Minn. (E. H. Dresser, Pres.)

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PONS, FRANCISCO, Santurce (nr. San Juan), P. R.

PORTER, J. M., Easton, Pa.

POST & MCCORD, 101 Park Ave., New York, N. Y. (Andrew J. Post, Pres.)

POWELL, FRANCIS C., 64 Pleasant St., Dorchester, Mass.

POWELL, F. E., 25-27 Ferry Bldg., Auckland, N. Z.

POWER, S. M., School of Mines, Bendigo, Victoria, Australia.

POWERS, JOHN M., 609 W. 3rd St., Sterling, Ill.

POWERS & SON, EUGENE S., 1520 W. Locust St., Philadelphia, Pa. (E. S. Powers.)

POWERS KENNEDY CONTRACTING CORP., 149 Broadway, New York City. George C. Bingham.)

POZZO, ALBERTO, Corso Re Umberto 63, Torino, Italy.

P'POOL & SON, A., 2716 25th St., Meridian, Miss. (A. O. P'Pool.)

PRATT, H. B., Antrim, N. H.

PRIESTER CONSTR. CO., 1006 Kahl Bldg., Davenport, Iowa. (O. F. Priester.)

PRINCE CONCRETE CO., 212 North 38th St., Camden, N. J. (G. R. Prince & Co.)

PRITCHELL, FRANK S., Concrete Laboratory, Raleigh, N. C. (N. C. State Highway Commission.)

PUCCI, ANGELO, 332 First St., Rochester, N. Y.

PURVES, JOHN, 914 Monadnock Block, Chicago, Ill. (Wells Bros. Const. Co.)

QUEBEC DEPT. OF PUBLIC WORKS & LABOR, Government Bldgs., Quebec, P. Q. (Ivan E. Vallee.)

QUEBEC DEPT. OF ROADS, Parliament Bldgs., Quebec, P. Q. (J. L. Baulanger.)

RABBERS, OSCAR A., 1619 Reed Ave., Kalamazoo, Mich.

RABER & LANG MFG. CO., Kendallville, Ind. (John E. Lang, Pres.)

RADER, B. H., Conway Bldg., Chicago, Ill. (Lehigh Portland Cement Co.)

RADIGAN, FRANK J., Court House, Jersey City, N. J.

RANDALL, FRANK A., 160 N. La Salle St., Chicago, Ill.

RANSOHOFF, V., Ideal Concrete Machine Co., Cincinnati, Ohio.

*RANSOME CONCRETE MACHINERY CO., Dunellen, N. J. (A. P. Robinson.)

RAYMOND, CHARLES, 120 St. James St., Montreal, Canada.

*RAYMOND CONCRETE PILE CO., 140 Cedar St., New York City. (H. P. Hamlin.)

RAYWID, LEO., 719 Quebec Place, N. W., Washington, D. C.

REAGAN, JAS. W., 202 N. Broadway, Los Angeles, Calif. (Los Angeles County Flood Control District.)

REBELLEDO, MIGUEL, 11a Artes 169, Mexico City, D. F., Mex.

REED CO., WILLIAM T., 200 Devonshire St., Boston, Mass. (William T. Reed.)

RENO CONCRETE FACTORY, 1026 W. 1st St., Reno, Nevada. (L. C. Bernasconi.)

REYNVAAN, A. J., 326 Swank Bldg.. Johnstown, Pa.

RHEINSTEIN & HAAS, INC., 21 E. 40th St., New York City. (A. Rheinstein.)

RHETT, ALBERT, H., 8 Hillside Ave., Summit, N. J.

RHODE, HOWARD, 1030 Hamilton St., Allentown, Pa. (Lehigh Portland Cement Co.)

RIB-STONE CONCRETE CORP., 2-3 Chamber of Commerce Bldg., Batavia, N. Y. (Geo. E. Priest.)

RICE, JAMES, P. O. Box 10, Forest Hills, L. I.
RICE, JOHN A., 1165 Arch St., Berkeley, Calif.
RICH, MELVIN S., 1410 H St., N. W., Washington, D. C.
RICHARDS, CLARENCE E., II, 584 E. Broad St., Columbus, Ohio. (Richards, McCarty & Bulford.)
RICHARDSON, JAMES H., Technology Club, 17 Gramercy Park, New York City.
RICHARDSON-JONES, HARRY, 1534 Magazine St., Honolulu, Hawaii.
RICHART, FRANK E., University of Illinois, 300 Lab. of Applied Mechanics, Urbana, Ill.
RICHMOND, KNIGHT C., 10 Weybossett St., Providence, R. I.
RICHMOND PATENT BUILDING BLOCK CORP., 612 Mutual Bldg., Richmond, Va. (G. Burgess, Pres.)
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RICKER, GEORGE A., Union Trust Bldg., Washington, D. C. (Dist. Engr. Portland Cement Assn.)
RIDDLE, JAMES H., Parkersburg, W. Va. (Dist. Engr., Portland Cement Assn.)
RIESCHE, ROBERT H., 514 Jackson St., Sioux City, Iowa. (Riesche & Sanborn.)
RITTER, LOUIS E., 140 S. Dearborn St., Chicago, Ill, (Ritter & Matt.)
RIVER ROAD SAND & GRAVEL CO., Merchantville, N. J. (Harry Chandler, Gen. Mgr.)
RIVET GRIP STEEL CO., 6014 Euclid Ave., Cleveland, Ohio. (G. G. Greulich.)
*RIVERSIDE PORTLAND CEMENT CO., 724 So. Spring St., Los Angeles, Calif.
ROBERTS & SCHAEFFER CO., 1110 Wrigley Bldg., Chicago, Ill. (E. E. Barrett, Vice-Pres.)
ROBINSON, ALBERT FOWLER, Room 1033, Railway Exchange Bldg., Chicago, Ill. (A. T. & S. F. R. R. System.)
ROBINSON, C. C., 1004-5 Times-Despatch Bldg., Richmond, Va. (Chas. M. Robinson.)
*ROBINSON, CO., INC., DWIGHT P., 125 East 46th St., New York, N. Y.
ROBINSON & CO., INC., DWIGHT P., 125 E. 46th St., New York City. (M. E. Thomas.)
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ROCKEFELLER, LLOYD H., Westmoreland St. Wharves, Philadelphia, Pa. (Pa. Brick & Tile Co.)
RODENBAUGH, H. N., St. Augustine, Fla. (Florida East Coast Railway Co.)
RODGERS, EBEN, c/o Alton Brick Co., Alton, Ill.
ROGERS, FLOYD, Newton, Iowa.
ROGERS-JENKINS & CO., Smith St., Box 1876, Durban, South Africa.
ROOS CO., THE H. W., 2036-46 Dana Ave., Cincinnati, Ohio. (H. W. Roos.)
*ROOS CO., THE H. W., Cincinnati, Ohio. (H. W. Roos, Pres.)
ROOS-MEYER-HECHT CO., THE, 2814 Stanton Ave., Cincinnati, Ohio. (G. W. Meyer, Secy.)

RORICK, C. L., 133 W. Washington St., Chicago, Ill. (The Permanent Bldr., Inc.)

ROSELAND CONCRETE PRODUCTS CO., 12110 So. Michigan Ave., Chicago, Ill. (W. C. Jones.)

ROSHOLT CO., THORMAN W., 421 S. 5th St., Minneapolis, Minn. (Thorman W. Rosholt.)

ROSS, FRANK B., 310 6th St., Laurel, Miss.

ROWE, HARTLEY W., 24 Federal St., Boston, Mass. (Lockwood, Greene & Co.)

ROWELL, W. A., Lakeport, N. H.

ROYAL SWEDISH BOARD OF WATERFALLS, Regeringsgatan 45, Stockholm, 3, Sweden. (Axel Ekwall.)

RUEBSAM, ERNEST C., 208 Union Trust Bldg., Washington, D. C.

RUSSELL, H. M., Riverside, Calif. (Riverside Portland Cement Co.)

RUST ENGRG. CO., 311 Ross St., Pittsburgh, Pa. (T. H. Wincherte.)

RYAN, WILLIAM R., 49 Wall St., New York City. (Thompson-Starrett Co.)

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*ST. MARY'S CEMENT CO., LTD., Room 14, 49 Wellington St., East, Toronto, Ont. (Geo. H. Gooderham, Pres.)

ST. PAUL CEMENT WORKS, 34 E. 4th St., St. Paul, Minn. (H. C. Berchem.)

SALE, PRENTISS D., JR., 1731 Columbia Rd., Apt. No. 402, Washington, D. C.

SAMPSON, GEORGE A., 83 Pembroke St., Newton, Mass. (Weston & Sampson.)

SANDERSON & PORTER, 52 William St., New York, N. Y.

SAUDQUIST & SNOW, INC., 323 Calumet Bldg., Miami, Fla. (Welton A. Snow, Chief Engineer.)

SANDSTROM, CHARLES O., 2931 Campbell St., Kansas City, Mo.

*SANDUSKY CEMENT CO., 626 Engineers' Bldg., Cleveland, Ohio. (William B. Newberry.)

SANTA CRUZ PORTLAND CEMENT CO., Crocker Bldg., San Francisco, Calif. (E. W. Rice.)

*SANTA CRUZ PORTLAND CEMENT CO., Crocker Bldg., San Francisco, Calif. (George R. Gay.)

SAUM, IRVING R., 3218 Newark St., N. W., Washington, D. C. (Wardman Constr. Co., Inc.)

SAUNDERS, W. L., 1812 G St., Washington, D. C. (Concrete Steel Co.)

SAURBREY, ALEXIS, 2112 Oliver Bldg., Pittsburgh, Pa. (Mellen-Stuart Co.)

SAVILLE, CHRISTOPHER JAMES, Portland, New South Wales, Australia. (Commonwealth Portland Cement Co.)

SCANLAN, J. A., 2323 Pioneer Rd., Evanston, Ill.

SCHEINDENHELM CO., EDWARD L., 111 W. Monroe St., Chicago, Ill. (Edward L. Scheidenhelm.)

SCHLEE, HERBERT J., 615 Wayne St., Detroit, Mich. (Truscon Steel Co.)

SCHMIDT, PAUL S., 4500 Euclid Ave., Cleveland, Ohio.

SCHNACK, BENNO J., Calle Mariano Pelliza, 886 Olivos, Buenos Aires, Argentina Republic.

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WRIGHT, EVERETT G., c/o Albert Kahn, Marquette Bldg., Detroit, Mich.

WRIGHT, NELDIEN H., Box 313, West Jackson Sta., Jackson, Miss.

WRIGHT & KOWALSKI, 15 Exchange Pl., Jersey City, N. J. (Donald A. Wright.)

WU, POND S., Supervisor of Works, Govt. Salt Administration, Peking, China.

*WYANDOTTE PORTLAND CEMENT CO., Ford Building, Detroit, Mich. (John W. Boardman, Vice-Pres.)

WYETH-SCOTT CO., THE, Manning St. and B. O. Railroad, Newark, Ohio. (W. D. Wyeth, Pres.)

YAKIMA PAVING CO., 324 Perkins Bldg., Tacoma, Wash. (Geo. Milton Savage, Pres.)

YANDELL & CONGER, E. Lafayette St., Jackson, Tenn. (C. H. Yandell.)

YEOMAN, R. C., 133 W. Washington St., Chicago, Ill. (Construction Materials Co.)

YINGLING, JR., CLINTON K., 1105 17th St., N. W., Washington, D. C. (Municipal Architects' Office.)

YORK PATENT BLDG. BLOX CO., Cor. Chestnut St. and P. R. R., York, Pa. (C. C. Spahr, Pres.)

YOUNG, CHARLES H., 822 McKnight Bldg., Minneapolis, Minn.

Young, Clarence R., 98 Hilton Ave., Toronto, Ont. (University of Toronto.)

Young, R. B., 8 Stracham Ave., Toronto, Ont. (Hydro Electric Power Comm.)

Young, R. N., Young Bldg., Allentown, Pa. (Lehigh Portland Cement Co.)

Young, R. W., Paullina, Iowa. (Paullina Con. Silo Co.)

Young, Thomas F., 8 Great George St., Westminster, London, S. W. 1, England.

Young, Wm. C., 1101 Center St., Wilkinsburg, Pa.

Youngstown Burial Vault Co., 546 E. Indianola Ave., Youngstown, Ohio. (E. E. King.)

Zeidler, John L., 401 Francis St., St. Joseph, Mo. (Zeidler Concrete Pipe Co.)

Zipprodt, Roy R., 410 Grain Exchange Bldg., Sioux City, Iowa.

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